Soil-pile interaction

Soil-pile interaction of pile in deep layered soil under seismic Excitation

By

Belete Dinku

A Project

Submitted in Partial Fulfillment of the Requirements for the

Master of Engineering

September, 2017

Declaration

This is to certify that this project entitled "**Soil-pile interaction of pile in deep layered soil under seismic Excitation**" submitted in partial fulfillment of the requirements for the award of the degree of M.Eng., in Geotechnical Engineering to the School of Graduate Studies, Addis Ababa Science and Technology University, through the Department of Civil Engineering and Construction Technology, done by Mr. Belete Dinku, ID.No. **GSR/095/07** is an authentic work carried out by him under our guidance. The matter embodied in this project work has not been submitted earlier for award of any degree or diploma to the best of our knowledge and belief.

Name of Student Belete Dinku Signature and date………………….

Name of Supervisor Dr . Melaku Signature and date………………….

Acknowledgement

I would like to express my special thanks of gratitude to my advisor as well as our Department Head who gave me the golden opportunity to do this wonderful project on the topic (**Soil-pile interaction of pile in deep layered soil under seismic Excitation**), which also helped me in doing a lot of Research and I came to know about so many new things I am really thankful to them.

Secondly I would also like to thank my parents and friends who helped me a lot in finalizing this project within the limited time frame.

Abstract

When lateral loads are applied on a pile, lateral deflection of the pile depends on the soil resistance and the soil resistance in turn depends on the pile deflection and this dependence is known as soil-pile interaction. Dynamic soil-pile interaction analysis has become an important field in civil engineering over the past years. Several major earthquakes that caused damage to buildings and other infrastructure have brought a lot of attention to response of pile foundations subjected to dynamic loading. When a pile is subjected to a seismic excitation, deformation of the pile is caused by the movement of surrounding soil with the passage of seismic waves (kinematic interaction) as well as the inertial forces applied by the superstructure due to its oscillation during the excitation (inertial interaction). But, in actual engineering practice, pile responses are calculated using the pseudo-static approach which considers only the inertial interaction effects, which essentially neglects kinematic interaction effects. The field observations of pile failures after seismic events have highlighted the importance of incorporating kinematic effects in the design process. Hence some codes such as Euro code states those kinematic effects should be considered during the pile design process. However, still there is no definite method to techniques to analyses pile foundations for seismic loads considering both kinematic and inertial effects. In this research Finite Element Method (FEM) is used as the analyzing tool over the most widely used “Beam-On-Foundation” method, due to the reliability of FEM in simulating and analyzing of soil-pile interaction problems. First the techniques were determined and incorporated in a three dimensional model developed using the general purpose finite element software ABAQUS to simulate the soil-pile system during a seismic excitation. The model was then extended to model the deep piles in multilayered soil profiles. For the investigation an actual soil profile was obtained from a site investigation. This consists of a deep marine sediment layer at the top of the profile and underlying soil layers with increasing stiffness and the scaled El-Centro were given to the soil-pile system. Also analyses were carried out varying the uppermost soft soil layer thickness to investigate the effect of soft soil layer thickness on pile behavior. Finally a parametric study was carried using soil profiles; with a deep soft soil layer. The analysis carried out show that the developed model has the capability of capturing important pile behavior under seismic excitations such as response due to kinematic and inertial interaction effects, effect of soil stiffness on pile behavior, deflection patterns and permanent deformations. It highlights that, input to the superstructure in seismic analysis should be modified depending on the soil-pile interaction effects, rather than using the original motion at the base of the structure which is the normal engineering practice. Moreover, analysis results show that pile behavior is unique and depends on many factors such as the nature of the soil profile it is embedded, soft layer thickness and properties of input motions. Furthermore, the developed model provides reliable techniques to simulate soil-pile interaction which can be used in actual engineering practice and also can be extended for further research purposes.

The analysis carried out show that the developed model has the capability of capturing important pile behavior under seismic excitations such as response due to kinematic and inertial interaction effects, effect of soil stiffness on pile behavior, deflection patterns and permanent deformations. It highlights that, input to the superstructure in seismic analysis should be modified depending on the soil-pile interaction effects, rather than using the original motion at the base of the structure which is the normal engineering practice. Moreover, analysis results show that pile behavior is unique and depends on many factors such as the nature of the soil profile it is embedded, soft layer thickness and properties of input motions. Furthermore, the developed model provides reliable techniques to simulate soil-pile interaction which can be used in actual engineering practice and also can be extended for further research purposes.

Keywords: soil-pile interaction, seismic excitation, kinematic interaction, inertial interaction, multi-layered soil, soft soil.

Table of content

List of Figures

Introduction

Background

Pile foundation is a part of a structural system that is used to carry and transfer the loads of the structure to a deeper soil or rock with a higher bearing capacity ,avoiding the shallow soil with low bearing capacity .piles are commonly used as foundation of tall building, bridges, Dams, transmission towers, earth retaining structures. Wharfs and jetties.in most situations, the primary function of pile foundation are to transfer the axial loads arising from the weight of the superstructure. There are however exceptions where the primary functions of the pile foundation is to resist lateral loads such as in wharfs and jetties. Even though the primary function is to transmit axial loads in most of the cases, every pile foundation has to withstand some lateral loads. For example pile foundations in tall buildings and transmission towers have to withstand wind forces whereas the pile foundation in bridges wharfs and jetties have to withstand wave action. In addition to these commonly acting lateral loads, earthquake is the major causes of lateral forces. An earthquake causes a horizontal shack of the ground which in turn causes lateral forces that a pile has to withstand.

When the piles are loaded axially, part of the load is transferred to the ground through the base of the pile as base resistance and part is transmitted through the pile shaft or skin friction fig 1.2. If the resistance forces exceed the limits, pile failure occurs causing an excessive vertical deflection. Unlike axially loaded piles, laterally loaded piles transfer the load to the surrounding soil mass through the lateral resistance of soil. when lateral loads are applied on pile, the pile tries to shift in the direction of the applied load, pressing against the soil in front of the pile fig 1.2 .this will generate compressive and shear stresses and strain in the soil. The total resistance acting across the pile shaft balances the external lateral forces. contrary to the failure mechanism of axially loaded piles, kinematics of laterally loaded pile is complex ,if the pile is short it will translate under the applied lateral load, this lateral deflection of pile depends on the soil resistance in turn depends on the pile deflection and this interdependence is known as ***soil-pile interaction***.

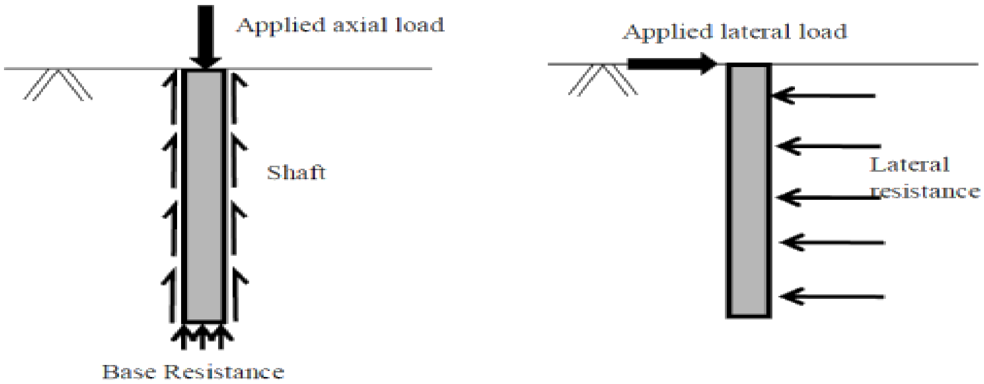


Figure 1.2: Load transfer mechanisms

The design of pile foundation on for seismic loads should take in to account the effect of the foundation on the ground motions and the effect of inertial loads imposed by the structure on the foundation.

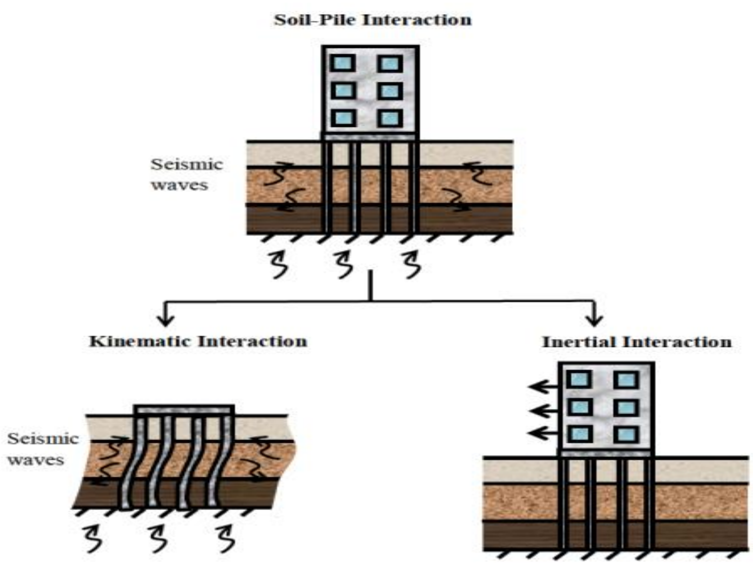


Figure 1.3: Components of soil-pile interaction

Dynamic soil-pile interaction analysis has an important field in civil engineering over the past years. Several major earthquakes that caused damage to buildings and other infrastructure have raised the interest on the response of pile foundation subjected to dynamic loading.

The soil-pile interaction consists of two components named as kinematic interaction and inertial interaction. The effect of the ground motion on the foundation is termed "kinematic interaction" and the effect of the ground motion on the foundation is termed “inertial interaction” fig 1.3

Design of pile foundations is still challenging, especially when they are embedded in soil profiles with soft clayey soil layers due to the lack of understating of their behavior under seismic excitations. Damages result in fatalities and interruption to services which will affect the day today life of the community as well the industries. Better understating of soil-pile interaction behavior under seismic excitations will lead to the safer design of structures and reduce the loss of lives and recovery costs of earthquakes.

1.2 Aims and Objectives

The aim of this project is to develop and apply a procedure to analyses pile foundations Embedded in multilayered deep profile strata comprising of soft soils when subjected to seismic excitation considering soil-pile interaction effects.

The specific objectives are:

- Develop a comprehensive three dimensional finite element model for analyzing soil-pile interaction process and hence provide a rationale method for the seismic analysis of pile foundations.
- Investigate the seismic performance of the interactive soil-pile system in deep multi-layered estuarine deposits.
- Study the influence of properties of soil layers and seismic records on the seismic response of the pile.

1.3 Method of Investigation

This research used the Finite Element Method as the analysis tool and the general purpose finite element software **ABAQUS** is used for the model development and analysis. This method of analysis was used in this research as it is considered as the most reliable method in representing the soil-pile system in three dimensional domains and also as it has the capability of modeling the behavior of soil continua.

First suitable finite element techniques were determined to represent the pile and the surrounding soil, along with their mesh sizes, constitutive models to represent material behavior, soil-pile interface, loading steps, damping, boundary conditions and the representation of the superstructure. A three dimensional finite element model was developed in the present study and was first validated using the existing results from the literature. The model with a homogeneous soil profile was then extended to one with a deep multilayered soil profile to investigate the soil-pile interaction behavior of deep piles embedded in multi-layered soil profiles with a soft soil layer.

1.4 Scope of the project

Soil profiles: The basic soil profile considered in this study was obtained from a site investigation. For the parametric studies, possible soil profiles were obtained by studying the site investigation reports

Pile foundation and Super Structure Loads: The pile sizes considered in this research are based on the standard sizes of precast concrete piles used in the civil engineering industry to support multi-story buildings. The super structure loads are also based on possible loads a precast pile may be subjected to when supporting a multi-story building.

Seismic Records: El-centro seismic records were used in the present study. The seismic records were scaled to have the same peak ground acceleration.

1.5 Layout of the project

Chapter 1: Introduction

Chapter 1 presents the background to the research topic, defines the research problem, states the aim and objectives and outlines the method of investigation of this research.

Chapter 2: Literature Review

Chapter 2 presents and overview of the soil-pile interaction behaviors, methods available to investigate the soil-pile interaction and the studies published in this area. It also highlights the Finite Element techniques applied to soil pile interaction behavior and the need for the present research.

Chapter 3: Development of a Comprehensive Finite Element Model

Chapter 3 presents the identification of suitable finite element techniques to simulate soil-pile interaction behavior and the development of a comprehensive finite element model to simulate the soil-pile systems.

Chapter 4: Application of the Developed Numerical Model

Chapter 4 presents the detailed study carried out to investigate the soil-pile interaction beha 1.6 Layout of project.

Chapter 5: Conclusion and Recommendation

Chapter 5 .states the main contributions of this research, conclusions drawn from the studies carried out and the recommendations for further research.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

The response of a structure subjected to a seismic excitation depends on the characteristics of the structure itself, mechanical properties of the surrounding soil, the interaction between soil, foundation and the structure and the seismic input. Traditionally, soil-pile interaction has been considered beneficial for seismic response of structures. After a rigorous analysis, Mylonakis and Gazetas [6] showed that the soil-pile interaction is not always beneficial and Kavvadas and Gazetas [7] have suggested that soil-pile interaction effects can increase structural demand. Traditionally building codes have not accounted for soil-pile interaction effects. However, the structural design code in the Eurocode series [5] includes the recommendations for foundation design for seismic loading considering soil-pile interaction. This chapter presents features of soil-pile interaction, methods available for soil-pile interaction problems, finite element techniques applicable for soil-pile interaction problems, and the knowledge gap in the subsequent sections.

2.2 Soil-Pile Interaction Analysis – Approach

Figure 2.1 shows the soil-pile interaction system and its key features. Since the forces that result from soil-pile interaction govern the structural response, these forces should be determined from accurate analyses. Soil-pile interaction can be carried out using two scenarios: either by modeling the structure and soil together with appropriate interface behavior as show in figure 2.1 or by using the principle of superposition as shown in figure 2.2. The superposition approach has two steps that address two different mechanisms, kinematic and inertial interaction as described in subsequent sections.

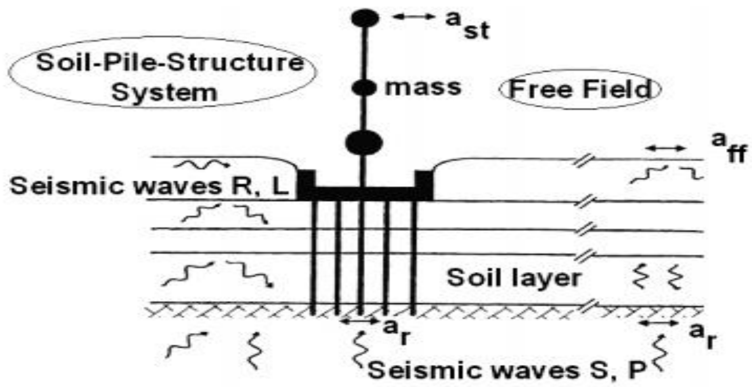


Figure 2.1: Soil-pile interaction system [8]

2.2.1 Kinematic Interaction

In the absence of the superstructure, as shown in figure2.2 a), the motion of the foundation may be different from the free field motion, where "free field" refers to the motion of the surface soil that is far enough from the foundation such that the foundation does not affect the free field motion. This difference is due to the kinematic interaction mechanism. The reasons for the observed differences are the presence of stiff foundations, wave inclination or incoherence or foundation embedment. Kinematic effects are described by frequency dependent transfer functions. The transfer function is defined by the ratio of the foundation motion to the free field motion in the absence of structure. Transfer functions are defined in the frequency domain. Wave passage through the foundation also generates stress in foundation elements. These stresses are termed "**kinematic stresses**".

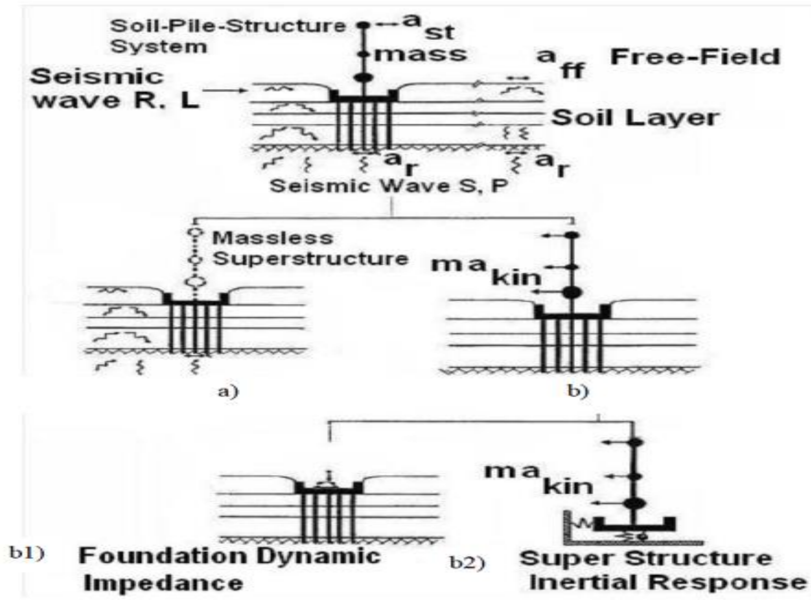


Figure 2.2: The superposition theorem for soil-pile interaction problem a) Kinematic interaction b) Inertial interaction [8]

2.2.2 Inertial Interaction

The motion at the foundation due to kinematic interaction forces the structure to oscillate. This in turn implies that the structure will produce inertial forces and overturning moments at its base. Due to this the foundation and surrounding soil will get additional dynamic forces and displacements. This is due to inertial interaction; the flexibility of the foundation support affects the acceleration within the structure. The flexibility of the foundation and the damping is associated with foundation impedance function (dynamic impedance).

The dynamic impedance can be simulated by the effects of a spring and dashpot acting at the base of the structure in place of the foundation elements. The above two mechanisms occur simultaneously with only a small time lag. In the two step approach, the acceleration at the top of the foundation is obtained by modifying the free field motion to account for kinematic effects. This motion akin is then used as an input motion for the analysis of inertial interaction. For computational convenience, the analysis of inertial interaction is further subdivided into two steps as shown in figure 2.2 b1) and 2.2 b2). First a dynamic impedance function at the top of the foundation is computed for the soil-pile system. As the final step, the superstructure, supported on the spring and dashpot system is analyzed using the akin as the input motion. The two-step method which uses superposition approach is based on the assumption that the system remains linear. Superposition is exactly valid for linear soil-pile and structure system [9]. However, superposition is approximately valid for moderately nonlinear systems under engineering approximations, because pile deformations due to lateral load transmitted from the structure vanish rapidly with depth.

Generally, kinematic interaction effects are neglected in structural design. This is acceptable in some circumstances such as at low frequencies [10] and for shallow foundations with vertically propagating shear waves or dilatational waves. However, Gazetas [11] carried out analysis on flexible piles with low frequency loading and concluded that the kinematic interaction is also important. In almost every seismic building code, structural response and foundation loads are computed by fixed base analysis neglecting soil-pile interaction effects.

2.2.3 Combined Kinematic and Inertial Effects

Earthquakes cause not only structural damage, but also geotechnical problems in buildings in the affected areas. Sometimes, structures supported on piles have settled and/ or tilted without a significant damage to their superstructure, when most of the time piles are embedded in weak soil profiles [4]. Field investigations and subsequent analyses have confirmed that both kinematic and inertial interactions should be properly accounted for in the seismic design of pile foundations. However, there is yet limited understanding on the combined kinematic and inertial interaction effects on the pile response during an earthquake. In order to enhance the understanding of the combined kinematic and inertial interaction effects on pile response some researchers have carried out experimental studies. Boulanger et. al. [12] carried out a series of centrifuge model tests which included two single piles with superstructure mass attached to an extension of the pile and subjected to nine different earthquake events with peak accelerations ranging from 0.02 to 0.7g. The soil profile consisted of soft clay overlying dense sand. Dynamic beam on foundation analyses were then carried out to evaluate the results from the tests. However, the authors have claimed that there is a 15%-20% deviation in test results and analytical results for peak superstructure motions and the peak pile bending moments along the pile length. Authors suggested that the differences are possibly due to approximations in the analyses that include the assumption of equivalent-linear soil behavior in the free-field, the uncoupling of site response and structural response, the use of independent p -y springs, and the uncertainties in soil properties and p -y characterizations. Also they have identifies the potential uncertainties or errors in the experimental data including the effects of soil-container interaction, influence of the pile foundations on the soil profile motions, limitations in the signal processing, and scale effect, very high levels of nonlinearity in the soil profile and around the piles. Tokimatsu et. al. [4] carried out a study to examine the effects of inertial and kinematic forces on pile stresses based on the results of large shaking table tests on pile-structure models with a foundation embedded in dry and saturated sand deposits. The soil profile in the test consisted of three layers including a top dry sand layer 0.5 m thick, a liquefiable sand layer 4 m thick and an underlying dense gravelly layer about1.5 m thick. A2 x 2 steel pile group that supported a foundation with or without a superstructure was used in this study. Each pile had a diameter of 165.2 mm with a 3.7 mm wall thickness, and their tips were connected to the container base with pin joints and their heads were fixed to the foundation that was embedded in the ground to a depth of 50 cm. the piles supported a superstructure of 139.3 kN. This study suggested that if the natural period of the superstructure is less than that of the ground, the ground displacement tends to be in phase with the inertial force from the superstructure, increasing the shear force transmitted to the pile. In contrast, if the natural period of the superstructure is greater than that of the ground the ground displacement tends to be out of phase with the inertial force, restraining the pile stress from increasing. However, it should be noted that this cannot be generalized for all the soil profiles and further investigations should be carried out considering different soil profiles. Due to the complex nature of the problem not many studies were conducted considering the kinematic and inertial combined effects. Most of the time it is not possible to carry out experiments for every situation in a laboratory environment and this can also be very expensive.

On the other modeling provides a reasonable method to predict the pile behavior under the combined kinematic and inertial interaction effects. However, it is important to establish the modeling techniques that can replicate the actual problem so that they could be used with confidence to extend the study and provide results with sufficient accuracy.

2.3 Analysis Methods

Soil-pile interaction has been a popular area of studies over the past few decades. As a result number of analysis methods is available to solve soil-pile interaction problems. However, all the analysis methods can be broadly classified as either beam-on foundation approach or continuum approach.

2.3.1 Beam-on-Foundation Approach

This concept was first started with representing shallow foundations that are long and flexible. In this approach, the beam represents the foundation and the foundation represents the soil mass. Winkler proposed that the vertical resistance of a subgrade against external force can be assumed to be proportional to the ground deflection [13]. Researchers have then extended this idea representing the ground with a series of elastic springs. The spring constants of these springs represent the stiffness of the ground against the applied load and the compression of the springs is proportional to the applied load. Due to the simplicity of the Winkler method, it is used widely and modified by many researchers later. This concept was further extended by placing an Euler-Bernoulli beam (to represent the actual foundation) on top of the elastic foundation (soil mass) (figure 2.3). A fourth order differential equation governing the deflection of such a beam-foundation was developed. Here, the input parameters are the elastic modulus and the geometry of the beam, the spring constant of the soil and the magnitude and the distribution of the applied load and by solving the equation, deflections, bending moments and shear forces can be obtained along the beam. It should be noted that there is a difference between these springs which are used to represent soil mass and the conventional springs. In conventional springs, spring constant multiplied by the spring deflection gives the spring force. But, in foundation springs, the spring constant multiplied by the spring deflection gives the resistive force of the foundation (ground) per unit length of the beam.

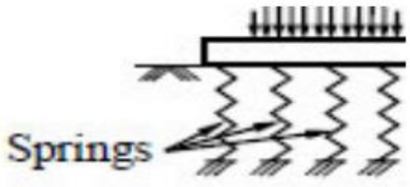


Figure 2.3: Beam on an elastic foundation

The beam-on-foundation method is also known as subgrade-reaction approach because foundation spring constant can be related to the modulus of subgrade reaction of a soil mass [14, 15]. If the pressure at a point on the contact surface (soil reaction) between the foundation and the beam is P and if the deflection of the point is y, then the modulus of subgrade reaction E s is calculated using the following equation.

$$\boldsymbol{E_s} = \frac{-\boldsymbol{p}}{\boldsymbol{y}} \quad \ldots \ldots\ldots \quad \text{Eq (2.1)}$$

The modulus of subgrade reaction multiplied by the width of the beam gives the foundation spring constant. The negative sign indicates the direction of soil reaction is opposite to the direction of beam deflection. In most cases piles behave as flexible beams when subjected to lateral loads and hence, beam-on foundation method was adopted by many researches to analyses laterally loaded piles (figure 2.4) by problem was looked upon by rotating 90 . Therefore the behavior of a single pile can be analyzed using the equation of an elastic beam supported on an elastic foundation [16], which is represented by the 4th order differential beam bending equation:

$$E_p I_p \frac{d^4 y}{dx^4} + Q \frac{d^2 y}{dx^2} + E_s y = 0$$

Eq.(2.2)

Ep = Modulus of elasticity of the pile

Ip = Moment of inertia of the pile

Q = axial load on the pile

x = vertical depth

y = lateral deflection of the pile at point x

Es = modulus of subgrade reaction

However, pile behavior under lateral loads is complex due to the nonlinear behavior of soils in real, particularly near the pile head. Therefore the linear springs suggested by Winkler were no more valid to describe the pile behavior under lateral loads and replaced by nonlinear springs, where spring constant changes with the pile deflection (beam-on-nonlinear-foundation approach).

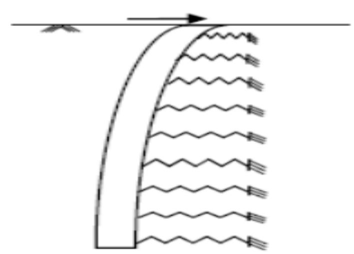


Figure 2.4: Laterally loaded pile with beam-on-foundation approach

The approached based on Winkler approach are the most crude approximations and hence used by many researchers to solve soil-pile interaction problems due to its simplicity. Novak [17] used one-dimensional Winkler model to simulate the dynamic soil-pile interaction. The analysis was done assuming linear elasticity of the soil-pile system and considering a viscoelastic medium holding a massless rigid cylinder subjected to a harmonic excitation. He has developed an approximate analytical approach which makes it possible to establish the dimensionless parameters of the problem and to obtain closed-form formulas for pile stiffness and damping. In his analysis, all components of the motion in a vertical plane were considered, i.e. horizontal and vertical translations and rotations of the pile head. The calculated dynamic stiffness and damping then used to predict the dynamic response of footings and structures supported by piles. Nogami and Novak [18] proposed a

Winkler model for the dynamic analysis of piles, based on continuum solution. In this method, soil medium around the pile shaft at any depth was idealized as a plane strain horizontal thin layer that is not coupled with horizontal thin layers at any other depths. Authors have argued that such an approximate soil model can give reasonable results under dynamic conditions compared to more rigorous solutions, in which three dimensionality of the soil medium and its dynamic interaction with the pile are accounted for. However, it fails to produce reasonable results at very low frequencies relative to the fundamental resonant frequency of the soil deposit. A modified version of Winkler model,which consists of a series of springs and dashpots (figure 2.5), was proposed by Nogami and Konagai [19]. This method was performed in time domain to calculate the flexural response of linear single piles. In this model, the soil mass was included through a soil radius, but the value of the soil radius was not addressed in the paper. The model could produce the dynamic response of single piles in a plane strain medium for a wide frequency range. Extending the aforementioned work, a nonlinear pile-soil interaction model applicable to both frequency and time domain dynamic lateral response analysis was proposed by Nogami et al. [20]. For the nonlinear dynamic analysis of soil-pile interaction, soil medium was divided in two regions; namely near-field and far-field. The near-field element was used to account for the nonlinear behavior of the soil in the vicinity of the pile shaft and the far-field element to account for the elastic behavior of the soil outside the region of strong nonlinear behavior (figure 2.6). In addition, an interface model was placed in between the pile shaft and the soil model in order to reproduce the formation and behavior of a gap at the soil-pile interface. However, the near field and the far field were separated artificially without any theoretical basis.

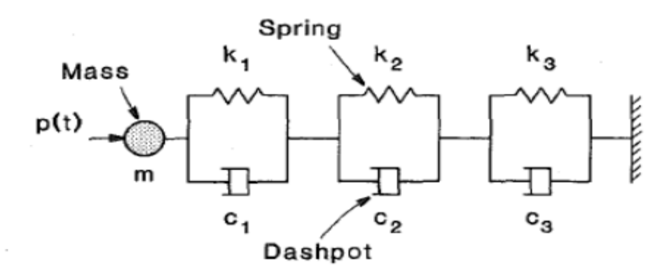
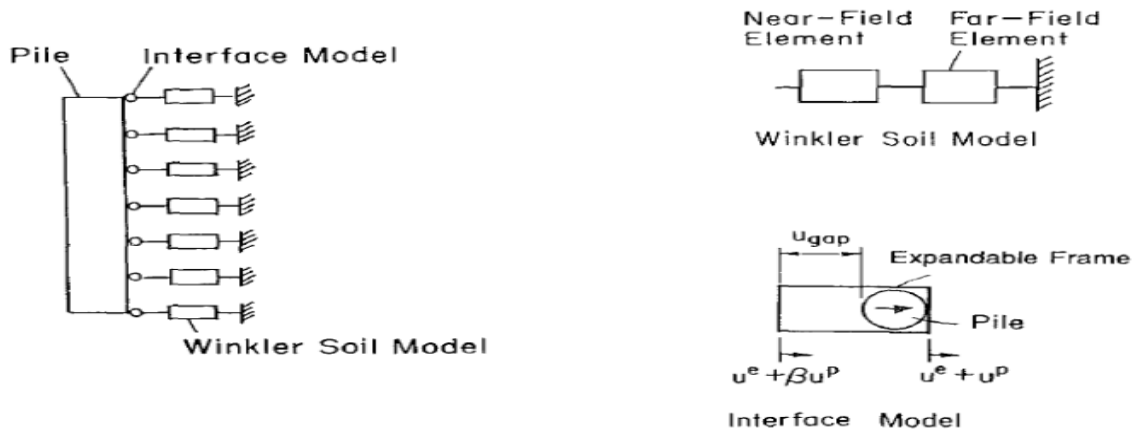


Figure 2.5: The modified Winkler model proposed by Nogami and Konagai [19]



a) b)

Figure 2.6: Schematic View of Soil-Pile Interaction Model proposed by Nogami et. al.[20]: a) Soil-Pile Interaction Model; b) Soil Model and Interface Model

Based on Winkler hypothesis, Naggar and Novak [21], proposed a method, in which soil reactions at both sides of the pile are modeled separately to account for the state of the stress and discontinuity at both sides as the load direction changes. In this method, pile was divided into segments with the same number and length as the soil layers. The analysis was formulated in the time domain to facilitate the modeling of the nonlinear behavior and discontinuity conditions and the elements of this model are shown in figure 2.6. The first part of the soil reaction model consists of inner field model, which consists of nonlinearity of soil. This consists of nonlinear springs, in which the stiffness is calculated with the assumption that plane stress conditions hold, the inner field is a homogeneous isotropic viscoelastic medium. The second part is far field model, which accounts for wave propagation away from the pile. Furthermore, the effect of neighboring piles is taken into account for piles in a group by introducing a viscos-elastic spring connecting the two piles through the far field as shown in figure 2.7. This model was then used by other researchers for the dynamic response analysis of piles and pile groups [22, 23].

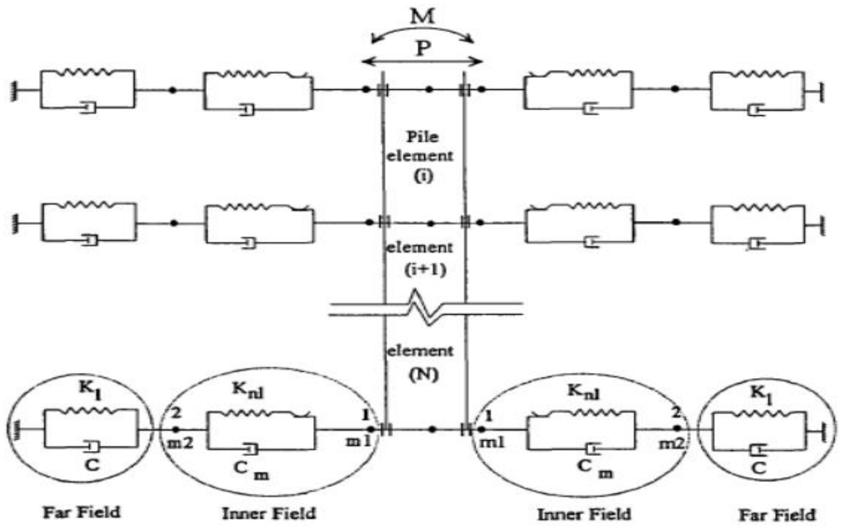


Figure 2.7: Elements of the method proposed by Naggar and Novak [21]

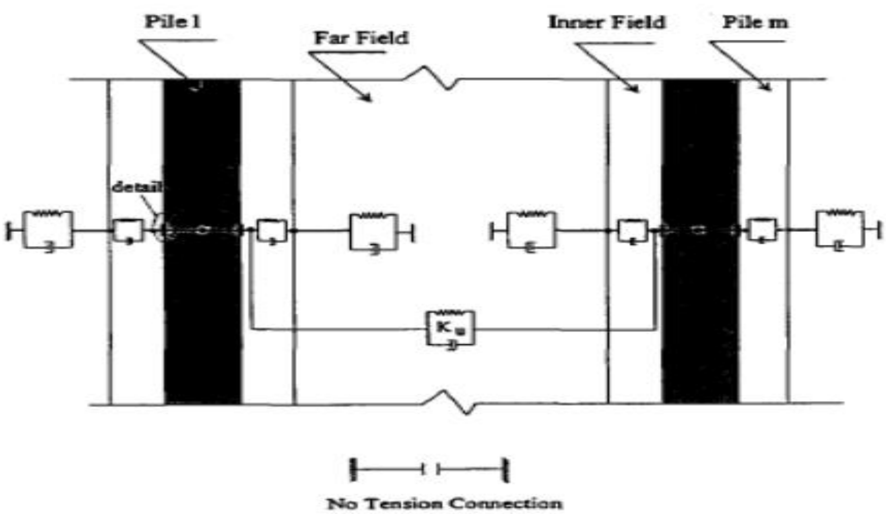


Figure 2.8: Elements of the model for group effect proposed by Naggar and Novak [21]

The aforementioned solutions for soil-pile interaction problems have considered only the kinematic interaction effects, but not considered the effects of inertial interaction effects caused by the presence of super structural mass. Liyanapathirana and Poulos [24] used the Winkler approach to determine the lateral response of piles in liquefying soils. In this study authors have considered the effect of superstructure on the pile response by attaching the superstructure mass at the cap level as shown in figure 2.8. To model the liquefaction of soil, this study has accounted for the reduction of soil stiffness and strength due to pore pressure generation and subsequent soil liquefaction in addition to the material nonlinearity.

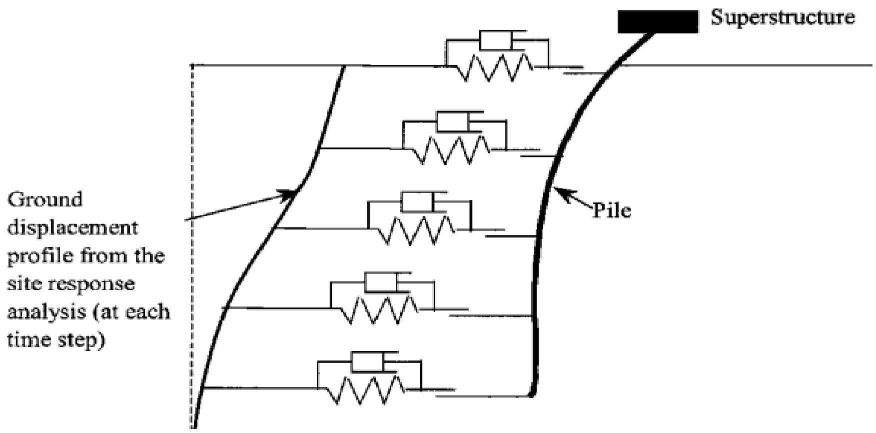


Figure 2.9: Winkler foundation method proposed by Liyanapathirana and Poulos [24]

Even though many developments were carried out to the traditional Winkler approach to simulate the dynamic behavior, such developments are not used frequently in practice due to its complex nature. However, as the subgrade reactions method (Winkler method) has a long history of use and also because of its simplicity in using the traditional way, this method is widely employed in practice in analysis of pile foundations under lateral loads. Despite of its frequent use, this method is criticized because of its theoretical shortcomings and limitations.

The main shortcomings are,

1. The modulus of subgrade reaction is not a unique property of the soil, but depends on pile Characteristics and the magnitude of deflection.
2. The method is semi-empirical in nature
3. The soil model used in the technique is discontinuous. The springs behave independently and the displacements at a point are not influenced by displacements or stresses at other points along the pile.

Beam-on-nonlinear-foundation approach was further modified and obtained the p-y analysis method [13]. In this method "p" stands for the soil resistance per unit length of the pile and y stands for the pile deflection. In contrast to Beam-on-nonlinear-foundation method where spring constant is considered as an input, p-y curves are given as inputs to the analysis in the p-y method. In the p-y method, soil is represented by a series of nonlinear p-y curves that vary with the depth and soil type. Therefore, these p-y curves are site specific and should be established specifically for each case. In this method, pile is divided into small divisions, and for each division a p-y curve is given as an input. To obtain the solution a fourth order differential equation (Eq. 2.3) should be used [25].

$$EI\frac{d^4y}{dx^4} + Q\frac{d^2y}{dx^2} - R - P_q = 0 \qquad \text{Eq.(2.3)}$$

Where,

Q=Axial load on the pile ; R = Soil reaction per unit length;

y = Lateral deflection of the pile at a point x along the length of the pile;

EI = Flexural rigidity ; Pq = Distributed load along the pile length

Depending on the magnitude of the deflection of the pile, the correct soil resistance should be calculated iteratively which satisfies the static equilibrium and achieves an acceptable compatibility between force and deflection (p and y) in every element. Shear, bending moment and slope can be obtained from the equations 2.4, 2.5 and 2.6 respectively [25].

$$EI\left(\frac{d^3y}{dx^3}\right) + Q\left(\frac{dy}{dx}\right) = S \qquad \text{Eq.(2.4)}$$

Where S = shear in the pile

$$EI\left(\frac{d^2y}{dx^2}\right) = M \qquad \text{Eq.(2.5)}$$

Where M = bending moment of the pile

$$EI\left(\frac{dy}{dx}\right) = S_t \qquad \text{Eq.(2.6)}$$

Where St = slope of the elastic curve defined by the axis of pile

The p-y method is considered as a versatile tool when compared to Winkler method, which usually produce reasonable results. Winkler methods ignore the soil's behavior as a continuum. If the continuum behavior is considered, the deflection at one point will affect the deflection at other points. However, there is no explicit method in p-y method also to incorporate the continuum nature of soils as it uses localized spring to represent soil. Nevertheless p-y curves are developed directly from results of load tests and the influence of continuum behavior is included indirectly which causes some unexpected results in some instances [26]. The accuracy of the p-y method depends on the number of tests and the variety of the tested parameters such as geometry and stiffness of the pile, layers of soil, strength and stiffness of soil and loading conditions [26]. One should be careful to extrapolate p-y curves to conditions where tests are not performed in similar situations. Even though the p-y method requires site specific measurements, it is considered as a versatile method, which provides a practical means for design. This method is used by American Petroleum Institute for the design of pile supported platforms and extended to design for offshore pile foundations. In many studies done on soil-pile interaction problems, where the pile is loaded laterally, soil has been considered as a linear elastic material. Hence these studies adopt linear springs to model the foundation (actual ground) in the beam-on foundation approach. One of the major disadvantages of the beam on foundation method is the two-dimensional simplification of the soil-pile contact which ignores the radial and three dimensional component of interaction.

2.3.2 Continuum Approach

In continuum approach, analysis of laterally loaded piles are done by treating the surrounding soil of pile as a three dimensional continuum in contrary to the beam-on-foundation approach. Therefore continuum approach is conceptually more appealing than the beam-on-foundation approach because the interaction of the pile and the surrounding soil is indeed three dimensional in nature. Poulos [27] has pioneered research in this direction; he has treated the soil mass as an elastic continuum and the pile as a strip which applied pressure on the continuum and the Mindlin's solution for horizontal load acting at the interior of an elastic half space and applied a boundary integral technique to obtain pile deflection. Even though many continuum based analysis methods are available, finite element method is the most versatile continuum based method of analysis used today. The other continuum based methods are less popular among the practitioners due to the complex mathematical analysis steps involved and do not provide simple, practical steps for obtaining pile deflections. In contrary, finite element method provides a comparatively convenient way of solving soil-pile interaction problems. Finite element method can take into account the three-dimensional interaction and both elastic and plastic behavior of pile can be simulated by giving inputs of Young's modulus and Poisson's ratio and by plugging in appropriate nonlinear constitutive relationships for plastic soils. One of the first applications of finite element analysis to piles was done by Yeigan and Wright [28] who introduced two-dimensional nonlinear soil models to analyses elastic piles. They have used that model to develop the lateral soil resistance – displacement relationships (p-y curves) for pile foundations. Since then some researchers have used finite element method to analyses piles [29, 30], mostly to verify the studies done by other researchers using different methods [31] and to obtain p-y curves. However, this method of analysis was not popular among researchers in early time as beam-on-foundation method. Even though finite element method claimed to provide the most powerful means for conducting soil-pile interaction analyses, it has not been used frequently until recent. The reason is that performing a three-dimensional finite element analysis requires a considerable amount of computational cost for generating input and interpretation results. However, with the advancement of technology, finite element method has become popular in soil-pile interaction analyses. A quasi 3-D finite element method was proposed for dynamic elastic and nonlinear of soil-pile interaction by Wu and Finn [32, 33]. The principle of the quasi 3-D model is shown in figure 2.12. This model was developed under the assumptions that shear waves in the XY and YZ planes governed the dynamic motions and the compression waves in the shaking direction Y and deformations were neglected in the vertical direction and normal to the direction of shaking. Dashpots were used to simulate the infinite soil medium. In this study, eight node brick elements were used to represent the soil and two node beam elements were used to represent the pile. In the model displacement compatibility between soil and pile was enforced. This model incorporated the soil yielding and gapping between the pile and the attached soil. An equivalent linear method was used to model the nonlinear hysteretic behavior of soil. Instead of varying the shear modulus with strain, a single effective value was used for the entire time history. In the single pile model, the superstructure mass was a rigid body and its motion was represented by a concentrated mass of at its center of gravity. A very stiff beam element with flexural rigidity 1000 times that of the pile was used to connect the superstructure and pile. In the group pile model, a concentrated mass at the center of gravity of the pile cap represented the rigid pile cap and mass-less rigid bars were used to connect the piles. The mass and pile heads were connected by very stiff mass-less beam elements. The authors have concluded that stiffness of the pile foundations decreases with the level of shaking.

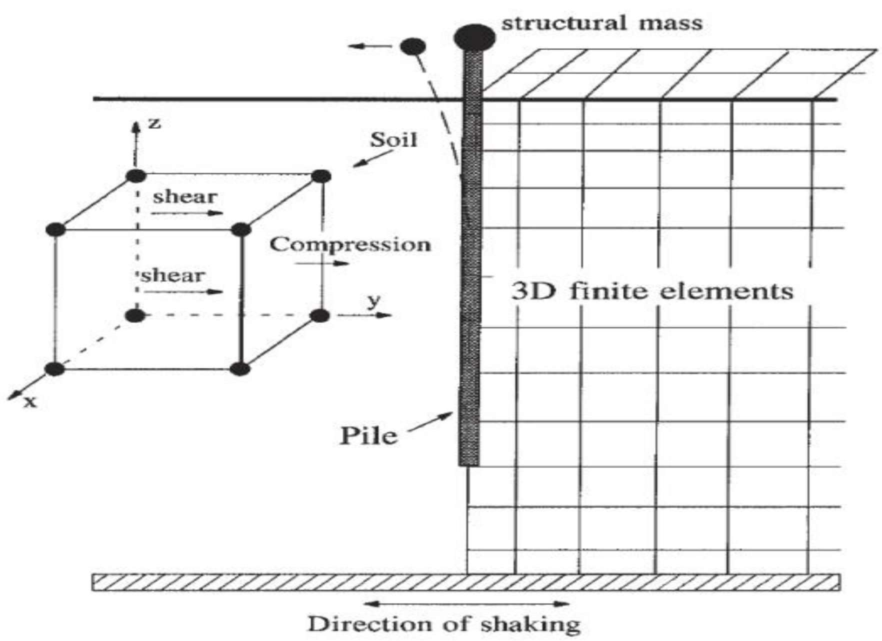


Figure 2.10: Quasi 3-D model for soil-pile interaction analysis [33]

Cai et al.[34] proposed a 3-D nonlinear finite element subsystem methodology. In this study, an advanced plasticity-based hierarchical single surface (HiSS) model was used to model the pile and soil.

Two node beam-column elements, which have six degrees of freedom for each node, were used to model the space frame of the concrete superstructure. Furthermore, eight node thin layers of soil isoperimetric elements with a HiSS constitutive law were used to incorporate the deformation modes of bonding, slipping separation and rebounding of the pile-soil interface. Depending on the refinement of the model, the pile may behave as linear or nonlinear. Kinematic and inertial interaction can be simulated simultaneously by using this model. Dynamic infinite elements were used to simulate and infinite medium and a recorded earthquake ground motion was used as bedrock motion. They concluded that a plasticity based soil significantly affects the pile foundation response from bedrock motion. Bentley and Naggar [35] studied the effects of kinematic interaction on the input motion at the foundation level. The 3-D model used in their study is shown in figure 2.13. In this study, they incorporated pile-soil separation, slippage, soil plasticity and 3-D wave propagation. By considering the symmetry one half of the actual model was developed in order to reduce the computing time. Kelvin elements were used to simulate the infinite soil medium. Soil was modeled as linear and elastoplastic material using the Drucker-Prager failure criterion. Liner elastic cylindrical piles were considered for this study. Two different types of soil-pile interfaces were considered either as perfectly bonded soil-pile interface and frictional interface. The Coulomb frictional model was used to incorporate the frictional interface behavior. Two recorded earthquake motions were used at the base of the model to simulate the seismic motion in the model. The authors have concluded that the elastic kinematic interaction for a single pile slightly amplifies the free field transfer function, i.e. the ratio of soil to bedrock motion.

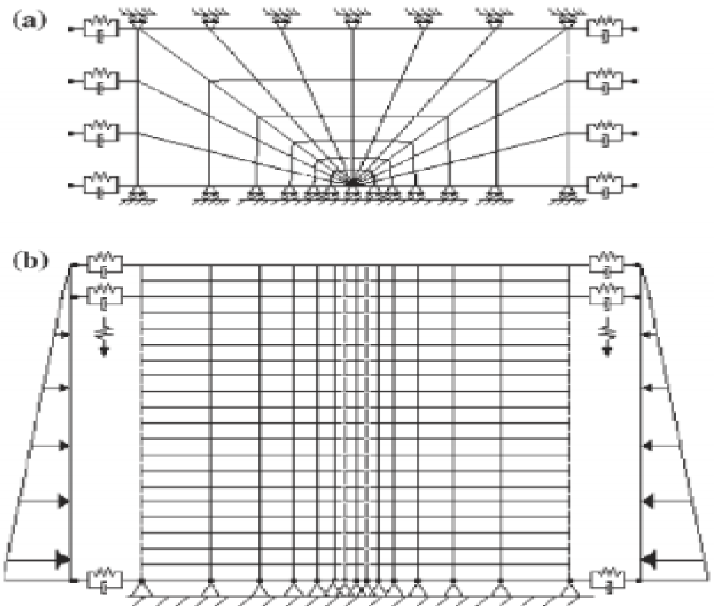


Figure 2.11: 3-D FEM model used by Bentley and Naggar [35] a) Plan view b) Front cross sectional view Maheshwari et al. developed a 3-D [36] finite element model to examine the effects of soil plasticity (including work hardening) and separation at the soil-pile interface on the dynamic response of a single pile and pile groups. The pile was modeled with a linear elastic material and the soil was modeled with an advanced plasticity-based hierarchical single surface (HiSS) model. Only one fourth of the model was constructed by considering symmetry and anti-symmetry. Kelvin elements (spring and dashpot) were used in all three directions (i.e. X, Y, and Z) to simulate the infinite soil medium. The model was loaded (at the base, which is assumed to represent bed rock) with the El Centro (north-south component) acceleration record from the 1940 El Centro Earthquake. Furthermore, harmonic motion was used to find the transfer and impedance functions for the foundation. Pile-soil separation was considered in the direction perpendicular to the motion. Friction between pile and soil was neglected. At every Gaussian point the normal stress in soil elements (in the direction of loading) and the confining pressure at corresponding depth were compared for every time step and at every iteration within a time step. Separation was assumed when tensile normal stress was higher than confining stress. The authors of this study have concluded that the effect of separation was more significant when using the elastic soil model rather than the plastic model. Also, nonlinearity reduced the real and imaginary part of the impedance function for the pile system. Moreover, they have concluded that the soil nonlinear response in the soil in the soil-pile system has significant effect for

low excitation frequencies. The authors have extended this study to account for inertial interaction effects by introducing separate superstructure and substructure systems [37]. In that study they have concluded that the inertial interaction increases the pile head response, but significantly decreases the structure response. They have also concluded that the soil nonlinearity increases both pile head and structure response at lower frequencies.

2.4 Finite Element Method Applied to Soil-Pile Interaction Problems

Since this study is focused on finite element modeling, the literature review presented in this section is focused on the instances where finite element method applied to soil-pile interaction problems and the corresponding modeling techniques.

2.4.1 General Modeling Details

Most of the studies which are found in the literature that used to solve soil-pile interaction problems using finite element method were based on 3-dimentional technique. Hence almost all the studies used eight node brick elements to model the soil [32, 33, 35-37]. However, piles were modeled using either 3-D beam elements [32, 33] or eight node brick elements [35-37].

2.4.2 Boundary Conditions

Unlike in static analysis the dynamic analysis of soil pile interaction using FEM considers surrounding soil strata as infinite in horizontal direction. In static analysis, the fixed boundary can be applied at some distance from the region of interest. However, in dynamic analysis, such boundary conditions will reflect outward propagating waves back into the model. Furthermore, fixed boundary conditions do not model adequately the outward radiation of energy at the boundaries of the model. A larger model can minimize this problem because material damping will absorb most of the energy in the waves reflected from finite boundaries. However, the increase in model size implies an excessive increase in computational time. To counteract reflections, some special non-reflecting boundary conditions have to be defined at the lateral boundaries. This will account for the fact that in reality the soil ought to be modeled as a semi-infinite medium. These types of boundary conditions are described in the following subsection.

2.4.2.1 Quiet Boundaries

2.4.2.1.1 Viscous Elements (Dashpot Elements)

Viscous elements were originally proposed by Lysmer and Kuhlemeyer [38] for the dynamic analyses of shallow foundations. The dashpot absorbs energy reaching the boundary. The dashpot coefficient per unit area in tangential and perpendicular directions to the boundary can be calculated from the following equations [38]:

$$C_n = \rho_s V_p \qquad \text{Eq.(2.7)}$$

and

$$C_t = \rho_s V_s \qquad \text{Eq.(2.8)}$$

Where, ps is the density of soil, is the Vp wave velocity, Vs is the shear wave velocity, Cn is the coefficient per unit area perpendicular to the boundary and Ct is the coefficient per unit area tangent to the boundary. However, authors claimed this method is applicable to only infinite systems for which all the disturbances and irregular geometrical features are limited to a small region of an otherwise homogeneous and linearly elastic space. Also this method cannot be adopted for the problems involving nonlinearities and transient loading conditions. Viscous elements are used by researches [32, 33] often in site response and soil pile interaction analysis. However, viscous elements do not provide stiffness to the model, which is the main drawback in simulating real scenarios.

2.4.2.1.2 Kelvin Elements

A Kelvin element consists of a spring and a dashpot attached in parallel (Figure 2.14). Kelvin elements can be attached to a boundary in order to simulate an infinite medium.The dashpot absorbs the energy that reaches the boundary, whereas the spring provides stiffness. Dashpot and spring constants can be determined using the solutions developed by different researchers, depending on the application [39]. This element is usually used to simulate the boundaries involved in both static and dynamic analyses. In static analysis, the damping term vanishes because of its dependency on frequency, since a dashpot absorbs energy as a function of velocity. When the velocity is zero, the dashpot term vanishes. Novak and Mitwally [38] developed a method to find the coefficients of the spring and the dashpot of the Kelvin element for a homogeneous or a composite (figure 2.15) media to harmonic, axis metrical dilation of a cylinder under plain strain conditions associated with the propagation of P-waves in the radial direction.

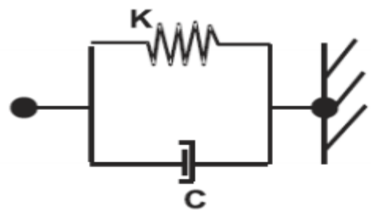


Figure 2.14: Kelvin element

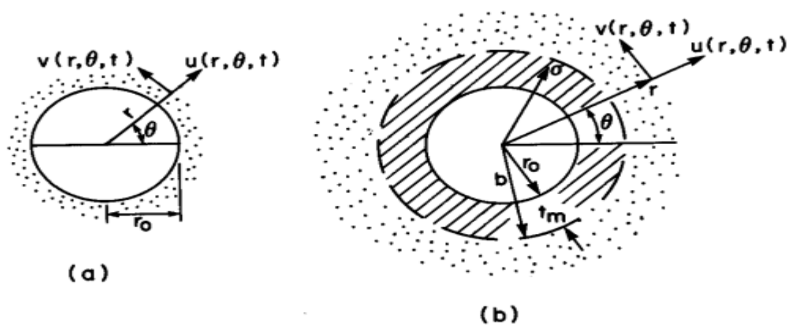


Figure 2.12: Notation for a a) homogeneous medium; b) composite medium [39]

However, in order to use these values the following conditions should be satisfied.

1. The medium is linear, homogeneous and isotropic with hysteretic frequency independent material damping

2. The cylinder from which the waves propagate is circular, massless and infinitely long and welded to the medium

3. The displacements are small and uniform along the cylinder

4. The vibration is harmonic

The constants of the spring and dashpot of the Kelvin element in the in horizontal direction was calculated using the equation [39];

$$k_r^* = \frac{G}{r_0}[S_1(a_r, \upsilon, D) + iS_2(a_r, \upsilon, D)] \qquad \text{Eq.(2.9)}$$

Where,

K_r^* = Complex stiffness

G = Shear modulus of soil

r_0 = Distance in plan from the center of the foundation to the node where Kelvin element is attached

S_1 , S_2 = Dimensionless parameters

D = Material damping ratio

υ = Poisson's ratio

a_r = Dimensionless frequency (= $r_o w/V_s$, where, is the angular frequency of excitation and Vs is the shear wave velocity of soil)

The real and the imaginary part of the above equation represent the stiffness (k) and damping (c_r) respectively, i.e.

$$k_r = \frac{GS_1}{r_0} \qquad \text{Eq.(2.10)}$$

$$c_r = \frac{GS_2}{\omega r_0} \qquad \text{Eq.(2.11)}$$

In a similar way Novak et al. [40] Developed a formula to obtain the spring and dashpot coefficients of a Kelvin element when the system is subjected to a vertical vibration under plain strain conditions. In this case also the same assumptions were made in obtaining the solution. Here, the spring and dashpot coefficients for the Kelvin elements in the vertical direction are calculated using the equation [40];

$$k_w^* = \frac{G}{r_0}\left[S_{w1}(a_r, D) + iS_{w2}(a_r, D)\right] \qquad \text{Eq.(2.12)}$$

Where subscript "w" is used to represent the vertical direction and the other parameters are same as in equation 2.9 Spring and dashpot coefficients are determined in a similar fashion to equations 2.10 and 2.11,i.e.

$$k_w = \frac{GS_{w1}}{r_0} \qquad \text{Eq.(2.13)}$$

$$c_w = \frac{GS_{w2}}{\omega r_0} \qquad \text{Eq.(2.14)}$$

Bentlet and Naggar [35] and Maheshwari et al [36, 37] used Kelvin elements in their analysis of soil pile interaction problems for single and group pile foundations.

2.4.2.1.3 Infinite Elements

Infinite elements are used in boundary value problems with unbounded boundaries (infinite medium) or in problems with a smaller region of interest compared to the surrounding medium. Infinite elements are usually used in conjunction with finite elements. The behavior of the infinite element is similar to that of the Kelvin element, but far nodes are not allowed to move. Infinite elements behave linearly providing stiffness dependent type of analysis In static analysis, stiffness is provided at the boundary based on the model of Sienkiewicz et al. , whereas in dynamic analysis is based on the model of Lysmer and Kuhlemeyer [38]. The dynamic response of infinite element is based on the assumption that the plane body waves travel orthogonally to the boundary. It is also assumed that the response adjacent to the boundary is of small amplitudes, so that the response of the medium is linear elastic. Wave propagation analysis of Zhao and Valliappan is an example for the application of infinite elements in dynamic problems.

2.4.2.2 Free Field Boundary

Free field boundaries are normally used to determine the response of site and pile foundations subjected to seismic excitation. Here, displacements at the lateral boundary are equal to that of the free field displacements. If the material damping of the soil is high, free field response can be achieved using a reasonably small distance from the structure to the edge of the model. However, when the material damping is low, free field responses are difficult to achieve with a limited distance from the model structure to the edge of the model. An alternative approach is to enforce the free field motion in such a way that boundaries act as an absorbing mechanism. This can be modeled by coupling viscous dashpots between main model nodes to soil column nodes at the edges, which represents the free field motion. The side boundary nodes of the main model and the soil column must have matching coordinates. However, this boundary condition only applies if the sides of the main model are vertical. This type of boundary condition has been used by researchers in seismic analysis of soil-pile interaction [8].

2.4.3 Soil-Pile Interface

Soil-pile interface modeling also contributes to the behavior of the soil-pile system. The soil- pile interfaces are usually modeled either as a perfectly bonded interface or as a frictional interface where soil-pile slipping and gapping may occur. In reality, the interface should be modeled to incorporate slipping and gapping. However, due to the high computational time and convergence problems, researchers consider a perfect bonding, if the problem to be analyzed is not dependent on slipping and gapping. Generally, Coulomb's law of friction is used to model slipping and gapping in FEM [8, 37]. If the interface is in full contact, full transfer of shear stress is ensured. Plastic slipping will occur when the friction stress exceeds the minimum of a user specified maximum shear stress or the friction stress due to the normal stresses at the interface. Separation will occur when there is tension between soil and pile interface. Besides the Coulomb friction model, there are other proposed interface models available in the literature.

2.4.4 Damping

If an un-damped system is allowed to vibrate freely, the magnitude of the oscillation is constant. However, in reality, energy is dissipated until the oscillation stops. In soil dynamics two kinds of damping properties can be estimated which decay the wave; namely, material damping and geometrical damping.

2.4.4.1 Material Damping

All materials possess a form of internal damping that makes them dissipate energy when deformed. Therefore as a wave spreads out from its source, the transmitted energy and the displacement and stresses induced at points far from the source will be dramatically reduced. Rayleigh damping is a form of material damping which is often used in mathematical models for the simulation of dynamic response of a system and it is proportional to the stiffness and mass of the structure [36]. This type of damping is represented by the following equation [37].

$$[C]=\alpha[M]+\beta[K] \quad \text{Eq.(2.15)}$$

Where,

[C] = Damping matrix of the physical system

[M] = Mass matrix of the physical system

[K] = Stiffness matrix of the physical system

a and b = predefined constants

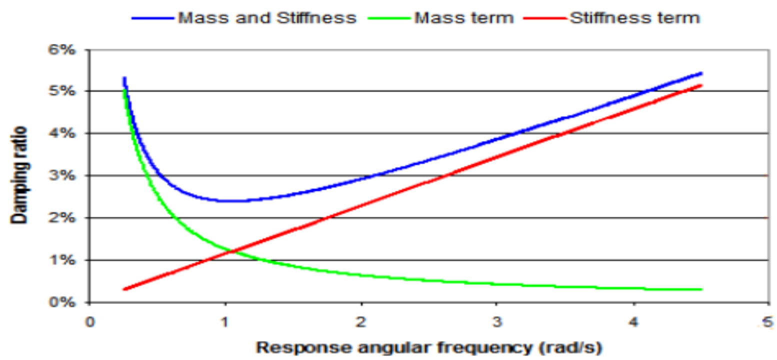


Figure 2.13 : Contribution of mass and stiffness damping terms to the overall damping ratio

Figure 2.16 illustrate the contribution of mass and stiffness damping terms to the overall damping ratio. Here, the stiffness proportional term contributes damping that is linearly proportional to response frequency and the mass proportional term contributes damping that is inversely proportional to response frequency.

2.4.4.2 Geometric Damping

Generally, a wave propagates equally in all directions meaning that the volume of material affected simultaneously by the wave, increases with the distance travelled by the wave. As a disturbance releases a fixed amount of energy, the energy absorbed by the medium per unit volume will decrease with such a distance. As a result, the amplitudes of the displacements and stresses induced by the wave will also decrease as the wave moves from its source. This type of damping due to the dispersion of wave energy over an increasing volume is known as geometric or radiation damping [25, 31].

2.4.5 Loading

In analyzing most structures, it is appropriate to begin with a complete mesh of stress-free unreformed elements and subsequently apply the specified loads to obtain the desired stress state. However, buried structures are an exception that their response depends on the history of the loading, i.e. In-situ state of stress in the ground. Therefore it is important to apply initial conditions before applying any external loads such as seismic loads. Several approaches are available to achieve these initial conditions [28]. One approach is to apply gravity loads to the structure and all the surrounding soil in the very first computational step. The external loads are then applied in the sub sequential computational steps. This method is simple to use, but may produce unrealistic and sometimes large tensile stresses in the soil. Soil deformations tend to be over predicted when this approach is employed. Another approach is to impose a user defined stress field onto the un-deformed soil mesh. Gravity is then applied in the first computational load step and displacements are computed to obtain force equilibrium. However, in this approach, quality of the final solution largely depends on the accuracy of the prescribed preliminary stress state.

After setting the initial conditions in the model, seismic loads can be applied. It can be applied either as a displacement, acceleration or velocity time history at the base of the model [35-37] or as a body force per unit volume (where ρ is density of the soil and is acceleration at base) distributed throughout the mesh [8].

2.4.6 Soil Behavior

Constitutive behavior of soil model is also an important aspect in soil-pile interaction analysis. Therefore, selection of a proper constitutive model leads to better results in Finite Element analysis. Generally, there are two types of soil models that are used in finite element analyses of soils. The first type consists of the elastic material models, of both linear and non-linear type. The second type is the elastic-plastic models such as Mohr-Coulomb, Drucker-Prager and Cam Clay. Models that have used linearly elastic as well as elastic-plastic [32, 33, 35] behavior to simulate the soil behavior of soil-pile interaction problems can be found in the literature. Even though complex constitutive models are available to simulate soil behavior, elastic-plastic constitutive model can provide a reasonable representation for a typical wave propagation problem. The aforementioned two types of soil models are explained in the following section.

2.4.6.1 Elastic Material Models

The basic assumption of elastic behavior is that the directions of principal incremental stress and incremental strain are coinciding. Elastic constitutive models can take the forms of isotropic or anisotropic and linear or nonlinear (Figure2.17). The isotropic elastic models involve two elastic stiffness parameters namely Young's modulus and Poisson's ratio. The linear elastic model is limited to the simulation of soil behavior. In reality, the stress-strain behavior of soil becomes non-linear, particularly as failure conditions are approached. Therefore, non-linear elastic models, in which the material parameters vary with stress and/or strain, are a substantial improvement over the linear models.

2.4.6.2 Elastic-Plastic Material Models

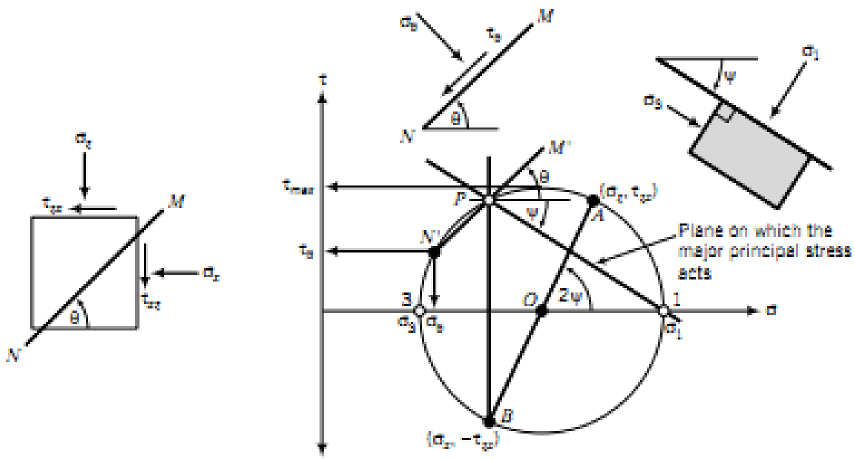
Elastic-plastic models provide a better representation of the real soil behavior. These models are based on the assumption that the principal directions of accumulated stress and the incremental plastic strain are coinciding. They require a yield function which separates elastic from elastic- plastic behavior and a plastic potential (or flow rule) which prescribes the direction of plastic straining. The two elastic-plastic material models which are commonly used to simulate the soil behavior are explained below.

2.4.6.2.1 Mohr-Coulomb Model

The Mohr-Coulomb plasticity model was proposed by Coulomb in 1773 for cohesive frictional materials. The yield criterion is expressed in terms of shear stress and normal stress acting on a plane. The model suggests that the yielding begins as long as the shear stress and normal stress satisfy the following equation;

$$\mathbf{T=C+s_n tan\varphi} \qquad \text{Eq. (2.16)}$$

Where, **c** is the cohesion and **φ** is the friction angle. The Mohr-Coulomb model is based on plotting of Mohr's circle for state of stress at failure in the plane of maximum and minimum principal stresses. The failure line is the tangential line to the Mohr's circle as shown in figure



The yield criterion of the Mohr-Coulomb model can be defined as:

$$f = (\sigma_1 - \sigma_3) - (\sigma_1 + \sigma_3).sin\phi - 2.c.cos\phi = 0 \text{ for } \sigma_1 \geq \sigma_2 \geq \sigma_3 \qquad \text{Eq.(2.17)}$$

Where s_1, s_2, s_3 are principal stresses, and s_1 and s_3 are maximum and minimum principal stresses (positive in tension). The Mohr-Coulomb failure model on plane and the Mohr-Coulomb yield surface on deviator plane.In terms of stress invariants and Lode's angle, (shown in Figure 2.20), the Mohr-Coulomb yield criterion takes the following form;

$$f = \sqrt{J_2} - \frac{m(\theta,\phi).sin\phi}{3} I_1 - m(\theta,\phi).c.cos\phi = 0 \qquad \text{Eq.(2.18)}$$

where,

$$m(\theta,\phi) = \frac{\sqrt{3}}{(\sqrt{3}.cos\theta + sin\theta.sin\phi)}$$

$$J_2 = \frac{1}{3}(I_1 + 2I_3)$$

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3$$

$$I_3 = \sigma_1\sigma_2\sigma_3$$

In the Mohr-Coulomb model, the plastic potential takes a very similar form of the yield function. In the plastic potential, instead of the friction angle, the dilation angle is used as follows:

$$g = \sqrt{J_2} - \frac{m(\theta,\psi).sin\psi}{3} I_1 - m(\theta,\psi).c.cos\psi = 0 \quad \text{Eq. (2.19)}$$

$$\text{where, } m(\theta,\psi) = \frac{\sqrt{3}}{(\sqrt{3}.cos\theta + sin\theta.sin\psi)}$$

If the flow rule is associated, then the yield criterion and the plastic potential coincides, which yields,

2.4.6.2.2 Drucker-Prager Model

The Drucker-Prager model was proposed by Drucker and Prager in 1952 for frictional soils. The yield criterion for the Drucker-Prager plasticity model is defined as,

$$f = q - p.tan\beta - c = 0 \quad \text{Eq.(2.210)}$$

where, $q = \sqrt{3J_2}$ is the generalised shear stress, $p = {}^{1}/_{3}\,(I_1)$ is the mean stress, β is the friction angle of the material and c is the cohesion of material. The parameter β and d can be matched with the Mohr-Coulomb material parameters c (cohesion) and ϕ (angle of internal friction). The Drucker-Prager yield surface is circular on the deviatoric plane and the three dimensional surface which is a cone is shown in figure 2.21. Figure 2.22 shows the comparison between the yield surfaces of Mohr-Coulomb and Drucker-Prager models.

2.5 Current design methodology of piles in seismic analysis in engineering practice

Pseudo-Static analysis is the most popular method of analysis of pile foundations under seismic excitation in engineering practice. In this method of analysis soil-pile system is broken down into two uncoupled systems, the superstructure and the foundation and then finding solutions to each that are compatible with the expected response of both parts [2]. In the first step of this analysis, the linear dynamic response of the superstructure is calculated by replacing the foundation with set of springs that represent the effective foundation stiffness to find the displacement demand of the superstructure. The pile foundation system is then analyzed using the push over analysis where the super structure is statically pushed to the displacement level established in the linear dynamic analysis step. The pile analysis is carried out using beam- on-foundation method. The Pseudo-Static methodology assumes that foundation is loaded primarily be the inertial loads caused by the superstructure. However, a pile foundation may also experience significant kinematic loads that are imposed by the surrounding soil mass as it deforms relative to the pile during a seismic excitation. Kinematic loading may not be significant in competent soil profiles with relatively small strains and deformations during an excitation. Nevertheless large kinematic forces can develop due to high strain in soft soils or due to lateral spreading of liquefied soils.

2.6 Gap

As explained in section 2.3 Beam-on-foundation methods have been widely used in soil-pile interaction analysis over the past few decades. This basic type of this method consists of springs attached at discrete locations to the pile to represent the surrounding soil which provides the resistance to the lateral movements when a lateral load is applied on the pile. However, many researchers have improved this method over the past decades to account for complex analysis such as soil nonlinearity, gapping, slipping and dynamic loading. However, this method of analysis is used mostly when load is applied at the head of the pile, where, kinematic forces of the surrounding soil do not affect the response of pile. On the other hand obtaining the spring constants under complex loading conditions such as seismic loads is not straightforward and does not provide a convenient method of analysis. Nevertheless the beam-on-foundation method's inability to simulate soil continua it is generally considered as inappropriate when it comes to seismic analysis of pile. In contrary, continuum method of analysis is considered as a sophisticated method of analysis when it comes to seismic analysis of pile foundations. Today, Finite Element Method is considered as one of the most convenient form of analysis in continuum method approach. It has the capability of simulating the soil continua under a seismic excitation unlike beam-on- foundation method. Furthermore, the three dimensional finite element analyses are more appealing because soil-pile interaction is indeed three dimensional in nature. But beam-on-foundation method is not capable of simulating this three dimensional nature of soil-pile interaction behavior. Despite of the high capabilities of Finite Element Method (FEM), it was not a popular method among researchers in analyzing soil-pile interaction behavior as it requires a high computational cost. However, with the advancement of the technology, it has become a promising method of analysis in the recent years. The limited studies carried out in using FEM are described in section 2.3 and these studies are based on homogeneous soil profiles with limited depth, typically around 10m. Most importantly these limited studies using FEM are mostly based on frequency domain analysis and cannot provide results with practical significance such as maximum deflections. In order to have such important results for pile design time domain analysis has to be carried out. However, no studies have done on modeling techniques that can be used for time domain analysis of pile and this research addresses this issue. Moreover, in real life situations, the soil profiles contains of layers with different stiffness's and piles are used to transfer to loads to deeper hard layers which may be located at 30m or so. Therefore time domain analysis of piles in such profiles can predict the behavior of piles during a seismic excitation which can be important during a pile design process. Furthermore, in actual engineering practice, pile responses due to kinematic forces are given minimum considerations. It may be reasonable if the piles are embedded in competent soil profiles which are not true for most cases.

2.7 Summary

This chapter presented the literature review carried out on soil-pile interaction behavior. This included the common methods of analysis of soil-pile interaction and the evolution of such methods, their advantages and disadvantages. As the present study is based on FEM of analysis, the FEM techniques applied to soil-pile interaction problems are then discussed. Finally the knowledge gap is described which lead to the present research on soil-pile interaction.

Chapter 3

DEVELOPMENT OF A COMPREHENSIVE FINITE ELEMENT MODEL

3.1 Introduction

In this study the general purpose of finite element software Abaqus is used to model the soil-pile system and investigate its behavior under seismic excitation. When such general purpose finite element modeling (FEM) software is used, it is important to select the proper finite element techniques, which closely resemble the soil-pile system and its behavior under dynamic loading conditions. The important components considered here are ; element that represent the soil and pile, mesh size ,constitutive models to represent material behavior of soil and pile, soil-pile interface behavior, damping, boundary conditions and loading steps.

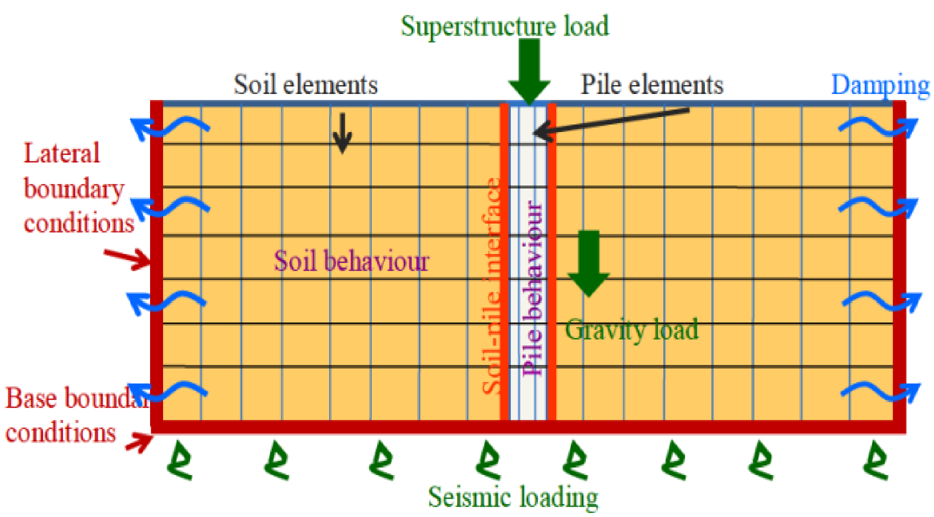


Fig 3.1 component of a FE model of the soil-pile system

3.2 Selection of the Finite Element Techniques

3.2.1 Elements

To model the pile, the conventional three dimensional brick elements (C3D in Abaqus) [54] were used. This type element has been used to model pile in past soil-pile interaction studies [35-37]. To model the surrounding soil, the same conventional three dimensional brick elements were used in most of the past research [35-37]. However, this conventional three dimensional brick element has limitations when used to represent soils and cannot fully simulate soils' actual behavior. In this type of element, lateral stress depends on the Poisson's ratio of the material of which the element is made of. But, in soils, the lateral stress distribution is governed by its internal friction angle. Also, under the applied gravity loading, deformations of the three dimensional brick elements become significant, and hence cannot replicate the actual in-situ conditions (figure 3.2). However, Abaqus provides a more sophisticated element type specifically to model soils, which is known as eight node tri-linear displacement and pore pressure element type (C3D8RP) [31] that overcomes the aforementioned problems. The suitability of this element type to model soil in contrast to traditional brick element type has been verified by real applications [55] and used in soil analyses successfull [54]. When soil is modeled with pore fluid elements, under the gravity loads, it shows negligible deformations (figure 3.3) and that resembles the real in-situ conditions.

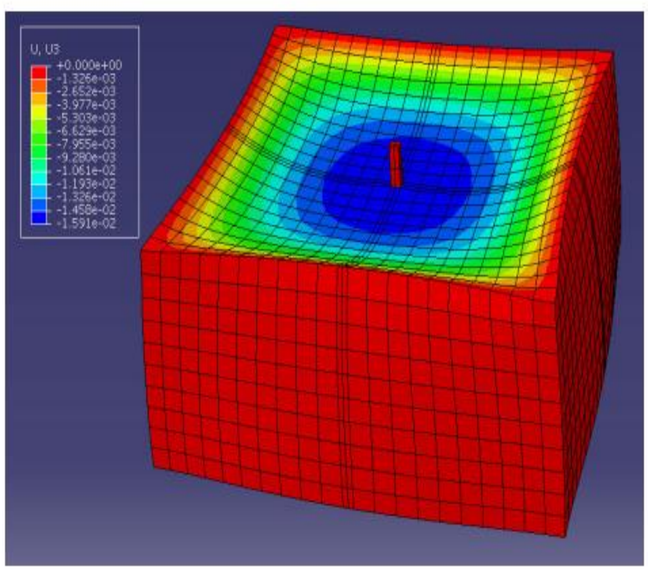


Figure 3.2: Significant vertical settlements in soil under gravity with traditional brick elements

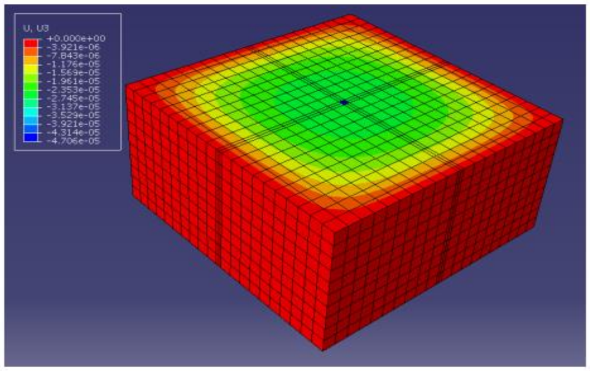


Figure 3.3: Insignificant vertical settlements in soil under gravity with pore pressure elements

3.2.2 Mesh size

The pile was considered as a cantilever beam fixed at the base for the purpose of determining the pile mesh size. A load was applied at the top of the column in the horizontal direction (figure 3.4) and deflections were obtained at different heights using FEM. To compare these numerical values theoretical values (Eq 3.1) [56] for the same scenario were calculated. The mesh size for which the deflections matched with the corresponding theoretical value closely was selected for use in further analysis (figure 3.5).

$$\mathbf{y} = \frac{Px^2}{6EI}\,(3l - x) \qquad \text{Eq. (3.1)}$$

Where, y= Deflection at the point considered

x= Height to the point considered from the base

E= Young's modulus of the pile

I= Second moment of area of the pile

l= Height of the pile

P= Applied load at the free end

The subdivisions in the vertical direction of the soil were kept constant within a soil layer to distribute the waves evenly in the soil profile. The maximum element size for soil was maintained at a value less than one-fifth to one-eighth the shortest wave length (l) to acquire the required accuracy [57]. Here, V_s ∕ f, in which V_s is the shear wave velocity and f is excitation frequency.

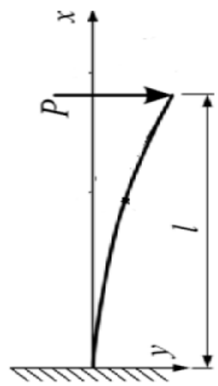


Figure 3.4 configuration for obtaining the column head response for pile mesh validation

3.2.3 Material models

Selection of proper constitutive models for the material behavior is important in numerical modeling. Similar to past research, this study also assumes that the pile behavior is linear elastic throughout the analysis [35-37]. Most of the past research on soil-pile interaction used elastic material models to simulate the soil behavior. But soil in most instances shows nonlinear behavior and hence plasticity should be incorporated. A simple elastic-perfectly plastic model can simulate the behavior of soil with a sufficient accuracy though there are different ways to incorporate the plastic behavior of soil. These types of material models have been successfully used in the literature in wave propagation problems [35]. The Mohr-Coulomb model which suggests that the yielding begins when the shear stress and normal stress satisfy the following equation was used in the present study.

$$\tau = \mathbf{C + tan\Phi} \qquad \text{Eq. (3.2)}$$

In the above equation, C is the cohesion and Φ is the friction angle of the soil. The yield criterion of the Mohr-Coulomb model is defined as:

$$\mathbf{f = (\sigma_1 - \sigma_2) - (\sigma_1 + \sigma_3).sin\ \Phi - 2C.cos\ \Phi = 0} \qquad \text{Eq. (3.3)}$$

Where σ_1 and σ_3 are maximum and minimum principal stresses.

3.2.4 Soil-pile interface

In ABAQUS, mechanical contact between two surfaces (bodies) can be modeled either as node based interaction or surface based interaction [54]. In node based interaction, mechanical contact between two nodes is modeled using contact elements, whereas in surface based interaction surfaces directly interact with each other. Surface based interaction has the advantage over the node based interaction because of its capability to model both normal and tangential interaction behavior whereas node based interaction facilitates only the normal interaction behavior. Surface based interaction has been successfully used to model soil-pile interface by researchers in the past [35, 58] and will be used in this study due to its advantages. This type of interaction in Abaqus [54] consists of the following steps.

1. Defining the surfaces which will be in contact
2. Defining the master and slave surfaces
3. Defining the mechanical (tangential and normal) properties of the surfaces

The two surfaces are to be defined based on their rigidities. The more deformable surface is defined as slave surface while the one with the greater rigidity is defined as the master surface. Master and slave surfaces for this study are surfaces of the pile and the soil respectively. The interaction behavior of these two surfaces was defined in terms of normal behavior and tangential behavior. Normal behavior was modeled as “hard” contact behavior. This approach allows any pressure to be transmitted between surfaces if they are in contact (Figure 3.6). The surfaces separate if the contact pressure reduces to zero. Separated surfaces come into contact when the clearance between them reduces to zero.

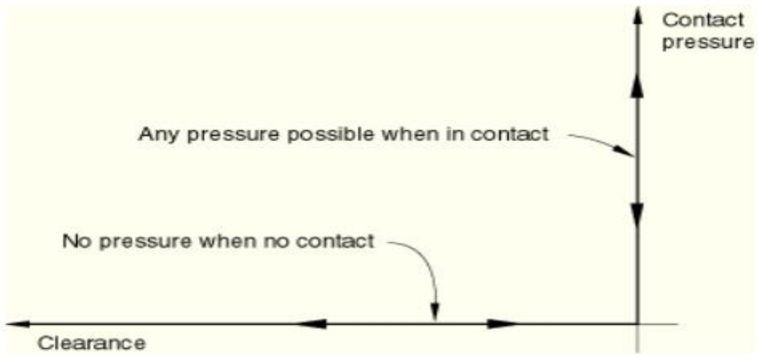


Figure 3.5: Hard contact behavior [54]

The tangential interaction behavior is based on the Coulomb friction model (figure 3.7). In this model, two contacting surfaces can carry shear stresses up to a certain magnitude across their interface before they start sliding relative to one another. The Coulomb friction model defines this critical shear stress, at which the sliding of surfaces starts as a fraction of the contact pressure, P between the surfaces ($\tau_{crit} = \mu P$).

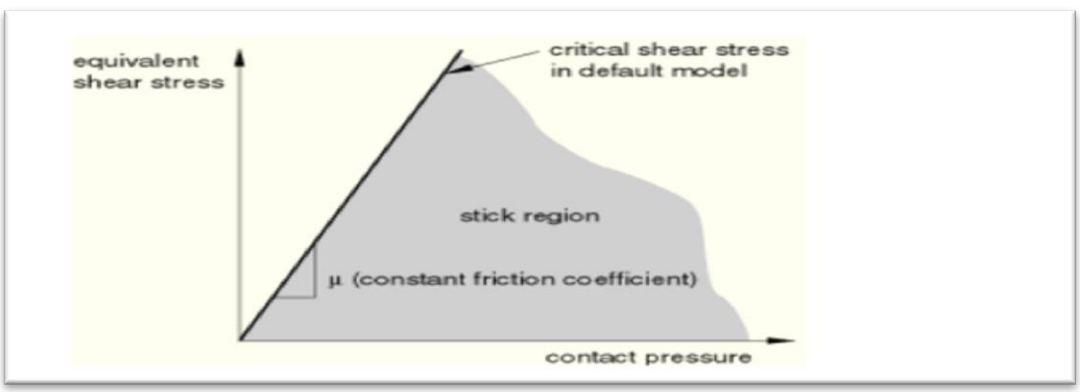


Figure 3.6: Tangential contact behavior [54]

3.2.5 Loading steps

Since the response of the pile foundation depends on the history of loading, it is essential to simulate in-situ stress conditions in the model before applying the seismic load. Therefore, prior to applying any seismic loading it is important to apply gravity loads and replicate the in-situ conditions. For this, gravity loads are applied in a separate computational step named Geostatic [54]. To avoid excessive settlements due to applied gravity loads, user defined stress field was applied to the soil mesh. However, in Abaqus, this procedure is allowed in the Geostatic step only if pore fluid elements are used to represent the soil. In defining the stress field, vertical stress at two points should be defined and the variation between those two points is considered linear. Here, vertical stress at a point (σ_v), is determined by considering the number of soil layers that lie above the point considered (n),

$$\sigma_v = \sum_{i=1}^{n} \gamma_n h_n \qquad \text{Eq. (3.4)}$$

where, γ_n = unit weight of the n^{th} soil layer

h_n = soil layer thickness of the nth layer with respect to the point considered

After defining the vertical stress distribution, the lateral earth pressure coefficient should be defined to calculate the horizontal stress (σ_h) distribution of the soil as follows.

$$\sigma_h = k_o \sigma_v \qquad \text{Eq. (3.5)}$$

Where, σ_h is defined as the lateral earth pressure coefficient at rest and calculated using following equation and the internal friction angle of soil (Φ) [59].

$$k_o = 1 - \sin\Phi \qquad \text{Eq. (3.6)}$$

When this step is invoked, stresses are calculated, which are in equilibrium with the external loading (in this case the gravity) and boundary conditions and produce zero or negligible deformations.

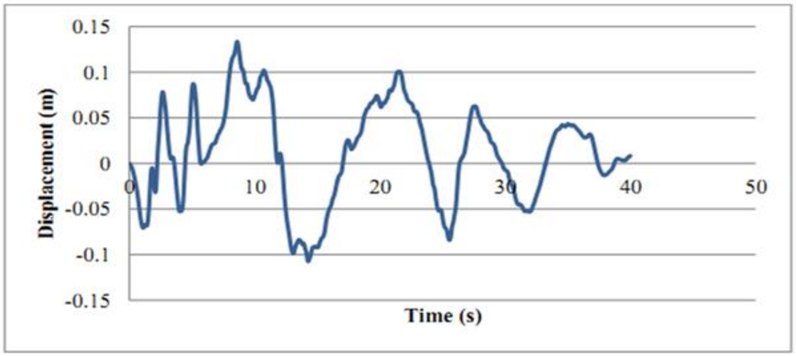


Figure 3.7: Displacement time history of El-Centro applied at the base of the soil-pile system [8]

A dynamic loading step is invoked after the Geostatic step in order to apply the seismic loading. Seismic load is applied at the base of the soil-pile system as a displacement time history at the bedrock level. This dynamic loading is applied in the horizontal direction and responses are measured in the shaking direction. A typical base motion applied in the present study is shown in figure 3.7.

3.2.6 Damping

In soil-pile interaction problems damping occurs in both the pile foundation and the soil. However, damping in pile is considered negligible when compared to that of soil. Due to this reason most of the studies conducted to investigate soil-pile interaction problems, did not consider the damping in pile foundation, but only the damping in soil [35-37]. This study also assumed that the damping occurs only in the soil, neglecting the damping in pile foundation.

In soil dynamics, damping is achieved through two scenarios namely; geometric damping and material damping. When a disturbance source releases some wave energy, the amount of energy absorbed by the surrounding medium per unit volume decreases as the wave travels away from the source. Consequently, the amplitudes of the displacements and stresses induced by the wave will also decrease as the wave moves from its source. This type of damping is known as geometric damping. In many soil-pile interaction problems with wave propagation, material damping of soils is represented by Rayleigh damping which was originally proposed by Rayleigh and Lindsay[60], in which the damping matrix results from the addition of two matrices, one proportional to the mass matrix and the other one proportional to the stiffness matrix as shown in equation 3.7

$$\mathbf{[C]=\alpha[M]+\beta[K]} \qquad \textbf{Eq. (3.7)}$$

Where, [C] = damping matrix, [M] = mass matrix, [K] = stiffness matrix

α, β = damping coefficients

However, selection of damping coefficients is challenging in soil dynamics and different authors have suggested different techniques to obtain these coefficients. These methods are complicated and were not used in the present study. Material damping in soils is considered to be achieved mainly through viscous damping. Therefore, traditionally, when computing material damping in soils, mass proportional damping is neglected and damping of the soil is achieved through stiffness proportional material damping. Damping matrix is hence reduced to a single matrix, which is proportional to the stiffness matrix as shown in equation 3.8

$$[C]=\beta[K] \qquad \text{Eq. (3.8)}$$

In this case, $\beta = \mathbf{2}\xi/\omega_{\mathbf{o}}$

Where, $\omega_{\mathbf{o}}$ is the predominant frequency of loading and is the material damping ratio which is assumed to be 5%. Predominant frequency is obtained from a Fourier spectrum drawn for the input wave as shown in figure 3.9. The frequency that gives the maximum Fourier amplitude is selected as the predominant frequency. This stiffness proportional damping was successfully used by other researchers [5,6] in Finite element analysis of soil-pile interaction problems and hence used in the present study as well.

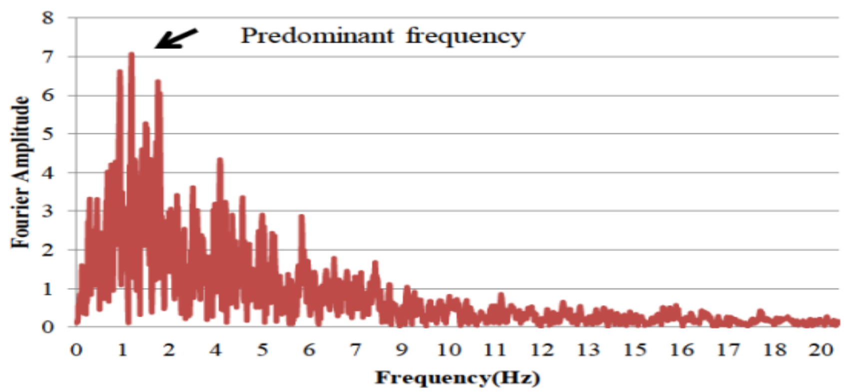


Figure 3.8: A typical Fourier transformations graph to find the predominant frequency of an earthquake wave

3.2.7 Boundary condition

In defining vertical lateral boundary condition for dynamic soil-pile interaction one of the important aspects is to avoid wave reflection at the vertical boundaries.to do so ,some researchers suggests that special boundary conditions which are known as “transmitting boundaries’ can be used.However.it is well known that even without using such special boundary conditions, the same effect can be achieved if the lateral boundaries are set at a considerable distance from the region of interests that the material damping will absorb most of the energy in wave reflected from lateral boundaries .but ,increase in computation costs associated with the increasing of model size is a main disadvantage in this type of boundary condition.

Different researchers have developed different transmitting boundary conditions’ to simulate the infinite extent of soil medium. Which is the actual condition of ground[38,39].in order to use these boundary types, such as wave types associated in the analysis, nature of the loading and material properties of the transmission medium as discussed in section 2.4.2.If this is not possible, those special boundary conditions cannot be used in the analysis.

To justify this argument, a simple analysis is carried out to simulate one-dimensional wave propagation under the free field conditions. A soil column of 10m height is modeled with an elastic modulus of 20MPa, density of 1203 kg/m and a Poisson’s ratio of 0.4. The damping ratio was assumed to be 5% and the soil was assumed to behave linear elastically throughout the analysis. Dashpots are attached at the lateral boundaries of the model as suggested by Lysmer [38], in perpendicular and horizontal directions with respect to the vertical lateral boundary with the dash pot coefficients calculated as described in section 2.4.2.1. Then a harmonic excitation was given at the base of the model with frequency of 1Hz and amplitude of 0.01m and the response at the top of the soil column was obtained. After that the inbuilt infinite elements [54] were attached at the lateral boundaries of the model instead of dashpots and the procedure was repeated to obtain the response at the top of the soil column. The amplification at the top of the soil column was then calculated by dividing the amplitude of the response at the top of the soil column by the amplitude of the input motion. The theoretical value of amplification in such a situation was then calculated using equation 3.9 as suggested by Gazetas [11] for one dimensional wave propagation.

$$\frac{Ug}{Uo} = \frac{1}{\cos(qh)} \qquad \text{Eq.(3.9)}$$

where,

$$q = \frac{2\Pi f}{Vs\sqrt{1+2iD}}$$

U_o = amplitude of the input bed rock displacement

U_g = amplitude of the free-field ground displacement

h = height of the soil stratum

f = frequency of the input motion

V_s = Shear wave velocity of the soil

D = damping ratio

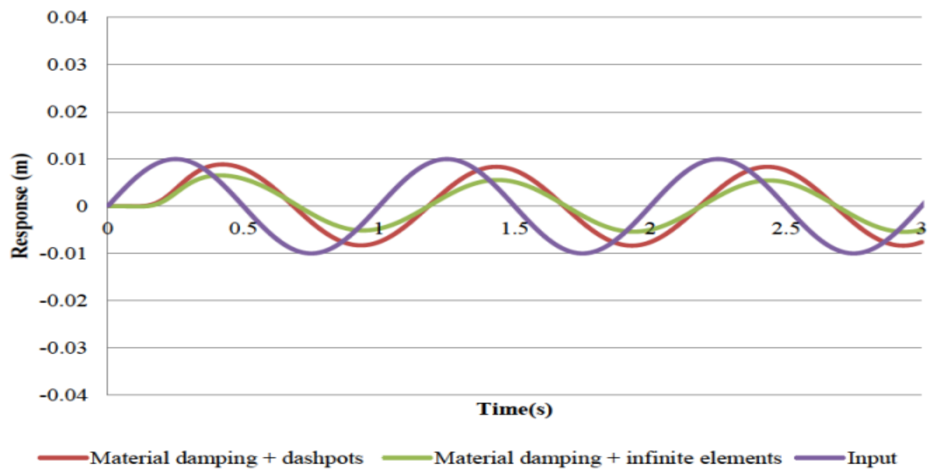


Figure 3.9: comparison of free field response for different boundary conditions

According to equation 3.9 that calculates amplification factor, at the frequency of 1HZ, a soil column with the given properties should give an amplification factor of 3.However, the obtained results show that (figure 3.9) simulation results do not give the required amplification due to over damping. As seen in figure 3.9 infinite elements in Abaqus are based on Lysmer dashpot diameters [54].

There are simple boundary conditions are suggested by some researchers [63,64],which can be used in dynamic soil-pile interaction problems under seismic excitation and are not required to satisfy any conditions. Example for such boundary conditions is. “Free horizontal and zero vertical’ ’and “repeated “boundary conditions [63]. In the present study “free horizontal and zero vertical” and “repeated” boundary conditions is adopted due to its simplicity. Moreover, this type of boundary conditions are used in commercial software [65] to solve problems associated with soils under seismic excitation .In this lateral boundary condition, horizontal movements are allowed while vertical movements are restricted. In this case, the static active failure of the vertical lateral boundaries should be prevented by applying lateral confining pressure at the boundaries (figure 3.10). However these lateral boundaries should be located at a sufficient distance from the area of interest to dissipate energy as much as possible, so that the reflected wave will not affect the response in the interested zone [65]. In this case a trial and error process was carried out to find out the location for the lateral boundaries, so that pile response is not affected furthermore with the change of the position. This lateral boundary condition was applied in the geostatic loading step and extended to the dynamic loading step.

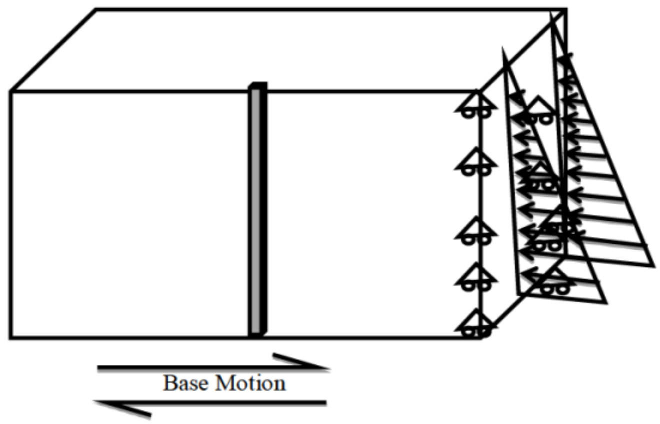
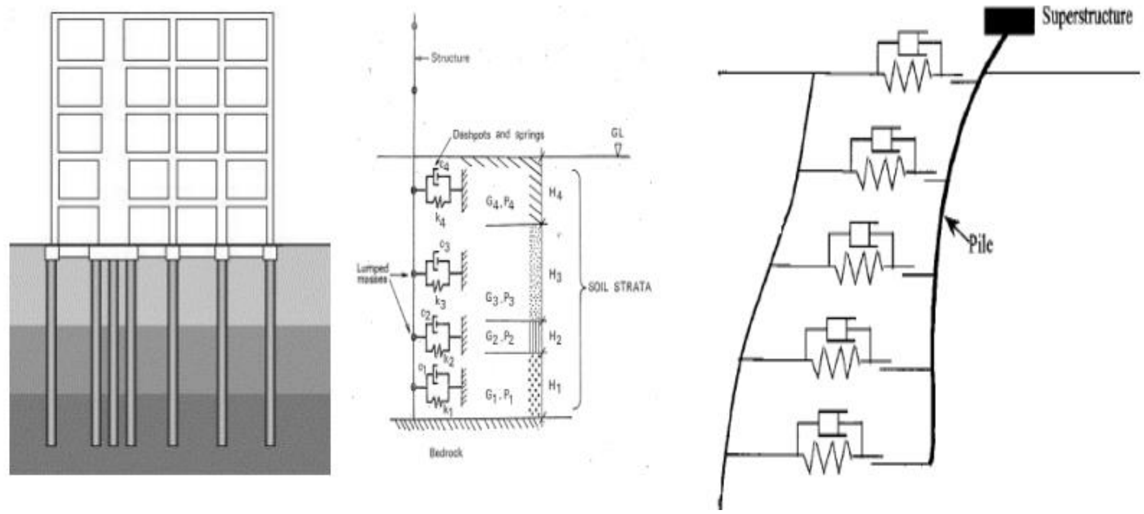


Figure 3.10 Boundary condition

Boundary condition for the base of the model depends on the loading condition. During the Geostatic loading step, the base was considered fixed. But in dynamic loading step, it is free to move in the horizontal directions. During this step the base was shaken in the horizontal direction.

3.2.8 Representation of the superstructure

Representation of the superstructure becomes challenging when it comes to real structures especially when it is massive. in some analysis carried out in soil-pile interaction problems, the whole structure is modeled on top of the pile(coupled system: modeling of the full structure founded on pile) (figure 3.11a [37,66].However when a multi-story building is considered such a modeling technique lacks practicality as it can increase the computational time and the cost drastically. In such situations, the common practice is to model the superstructure using lump-mass model. Even though the common tendency is to model the superstructure as a multi-degree of freedom structure with several masses attached at different levels (figure 3.11 b), Liyanapathirana and Poulos [24] suggested that attaching the superstructure mass at the cap level of the pile foundation provides sufficient accuracy for pile design (figure 3.11 c). Since the main idea of this study is to investigate the behavior of piles that support a multi-story building, the method suggested Liyanapathirana and Poulos [24] is used in the present study.



a) b) c)

Figure 3.11: Methods of representing the superstructure a) Modelling of whole superstructure b) Multi-degree of freedom superstructure c) Structural mass at the cap level

3.3 Summary

This chapter presented the methodology for developing a comprehensive finite element model that can be used to simulate soil-pile interaction. This included selection of appropriate element types for both pile and soil and determining their mesh sizes, material models to simulate the behavior of soil and pile, modeling of soil-pile interface behavior, damping, loading steps and boundary conditions.

This chapter also shows that, instead of using complex modeling techniques, simple techniques work well in simulating soil-pile interaction and the predicted results matched well with those from the studies done by using complex techniques and mathematical formulas. The developed modeling techniques validated using a homogeneous soil profile are then extended to investigate the soil-pile interaction in deep multilayered soil profiles with a soft soil layer and are described in the subsequent chapters.

Chapter 4

APPLICATION OF THE NUMERICAL MODEL

4.1 Introduction

Current practice of determining seismic moments and forces in pile foundations are done using pseudo-static approach where, base moments and shear forces are applied to the pile head and a static analysis is carried out using a Winkler spring model to simulate the interaction between pile and soil. However, this type of analysis neglects the effect of seismic shaking on pile response which is termed as kinematic interaction between pile and soil.

As the seismic waves pass through the soil layers, the soil layers move laterally and the piles are also forced to move with the surrounding soil media. In such a scenario, pile head and pile tip may move in different directions causing different deflection modes in the pile. On the other hand most of the studies carried out incorporating the transient nature of the earthquake loading are based on the Winkler approach that uses springs to simulate the soil-pile interaction. However, Winkler approach is claimed to be not reliable for seismic response analysis. On the contrary, a continuum based analysis such as the Finite Element Method can be used to simulate the kinematic pile behavior under a seismic excitation. For this, time domain analysis should be carried out using a three dimensional soil-pile system. Furthermore, most of the studies carried out in the area of soil-pile interaction are based on homogeneous soil profiles and the behavior of a pile foundation embedded in a deep multilayered soil under seismic excitation is not well understood. Even though the field observations after a seismic event provide some details about the behavior of piles in such situations, numerical simulations enable the investigation of pile behavior under a seismic excitation, as performing experiments are not feasible. However, only a limited number of studies have been carried out considering deep foundations embedded in multilayered soil profiles, probably due to the high computational cost and the skills required to carry out numerical simulation using Finite Element Analysis which is a very viable method to capture the real behavior of soil-pile interaction in three dimensional domain. This chapter describes the study carried out to investigate the seismic response of a pile embedded in a real (existing) soil profile, by extending the developed and validated numerical model. It will simulate the behavior of the pile foundation embedded in a deep multilayered soil profile with a soft soil layer at the top, as typically found in marine environments.

4.2 Soil Profile Data

This soil profile consists of 5 layers with stiffness increasing with depth. The soil layer thicknesses are 16m, 6m, 2m, 2m and 7m from top to bottom. The obtained soil properties are listed in table below.

Layer no	**Layer thickness (m)**	**Density (Kg/m^3)**	**Young's modulus (MN/m^2)**	**Poisson's ratio**	**Friction angle(dgree)**	**Cohesion (kN/m^2)**
1	16	1631	10	0.4	0	39
2	6	1835	15	0.4	0	59
3	2	1886	21	0.4	0	83
4	2	1937	63	0.3	35	0
5	7	1937	248	0.3	50	0

4.3 Selection of Earthquakes

Generally earthquakes have different characteristics with respect to level of shaking, dominant frequency, duration of strong motion, and duration of excitation and so on. In the present study, (El-Centro,) |were selected to be used. As far as the original earthquake records are considered, the El-Centro has maximum accelerations of 0.3g. In Australia, only few earthquakes have been recorded so far with Tenant Creek earthquake being the largest earthquake with a maximum acceleration of 0.35g. Therefore to simulate a credible earthquake and to facilitate meaningful comparison of results, all the earthquakes were scaled to have a maximum acceleration of 0.3g as shown in figure 4.1.

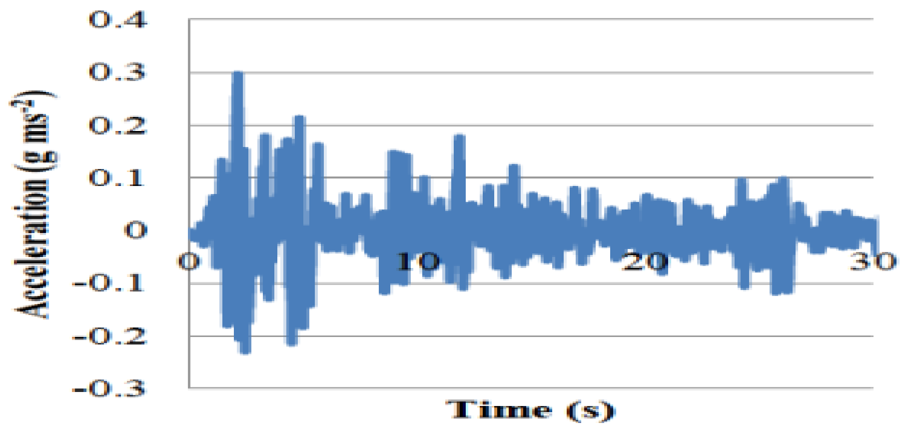


Fig 4.1 Scaled earthquake record El-centro

The El-centro earthquake has a considerable amount of shaking for a relatively longer period. In order to find the dominant frequencies a "fast Fourier Transformation" analysis was carried out for each input motion and the dominant frequencies for each seismic record as shown.

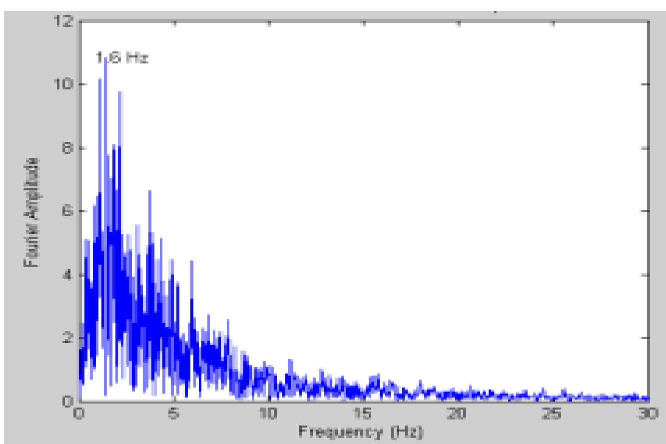
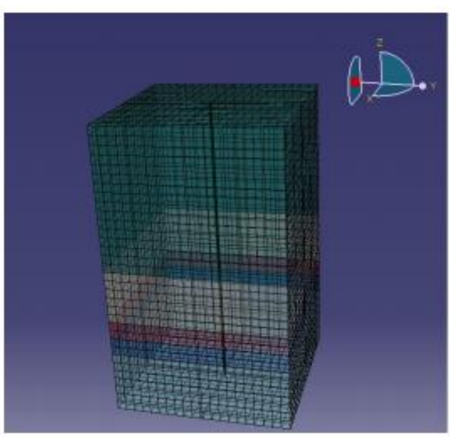
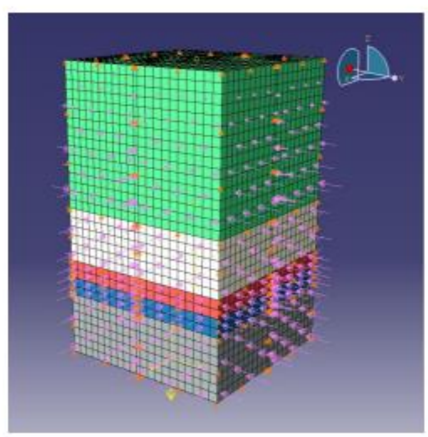


Figure 4.2: Fourier Transformations of the Earthquakes El-Centro earthquake

4.4 Model Development

To simulate the behavior of a pile foundation embedded in multilayered deep profile strata, during a seismic excitation, a precast concrete pile of 0.25m x 0.25m that runs through the whole 33m up to the bedrock was used to investigate the behavior. This is a standard for precast piles and used in the industry to support multi story building extensively. Young's Modulus and Poisson's ratio of the pile were taken as 36GPa and 0.15 respectively and the soil properties used here are listed in table 3.1. In this analysis, soil was considered as an elastic-plastic material, whereas pile was assumed to behave linear elastically throughout the analysis. Soil-pile FE model was developed using the modeling techniques explained in Chapter 3 (Figure 4.3). Unlike the homogeneous soil profile used in chapter 3, this analysis considers a layered soil profile. Therefore, the soil was divided into strata according to the thicknesses found from the CPT test results and as listed in the table 3.1 and each stratum was assigned the properties as found from CPT test results. The pile was considered socketed at the base and the scaled seismic excitations were applied at the base of the soil-pile system. In the ABAQUS model, seismic excitation is given as the displacement-time history (abaqus example) of the corresponding seismic record.





a) b)

Figure 4.3: Screen Shots of the Developed Model a) Pile embedded in the layered soil profile b) Soil-pile system with the lateral boundary conditions .

4.5 Pile Head Response

4.5.1 Kinematic Interaction applied to Soil-Pile System in the Time Domain

Figure 4.4 shows the pile head response when subjected seismic excitations. When subjected to the El-Centro earthquake, the pile head response follows the pattern of input motion where, peak response occurs near the peak input. In general, when the soil-pile system is subjected to El-Centro earthquake, the pile head response shows an amplification of three times the input motion, giving a maximum response at about 0.3m.

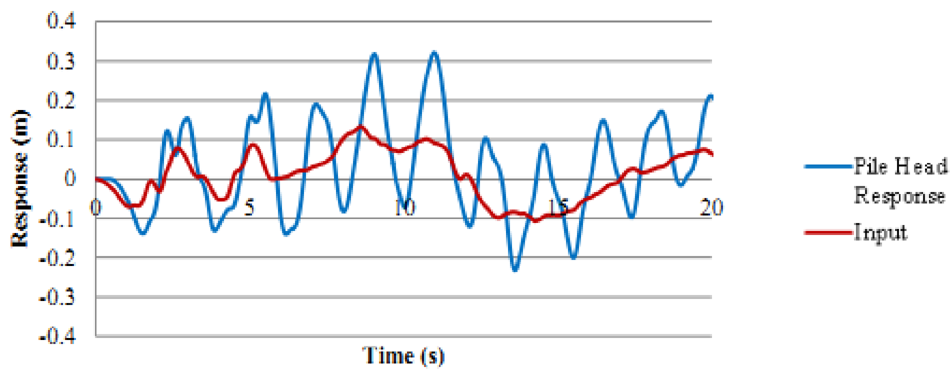


Figure 4.4: Pile Head Response when subjected to El-Centro Earthquake

4.5.2 Combined Kinematic and Inertial Interaction applied to Soil-Pile System in the Time Domain

In real life engineering application, piles are used to transfer the superstructure loads to deeper stiff soil layers. Therefore, piles are required to support the superstructure mass, can affect the pile behavior under a seismic excitation. In a pseudo-static analysis which is currently used in the seismic analysis of pile foundations, only the inertial interaction effect caused by the structural mass is considered and hence it cannot capture the actual behavior of the pile under a seismic excitation. This section of the chapter presents the behavior of the pile head response under the combined effects of kinematic and inertial interactions. This is an important aspect in designing not only the pile, but also the superstructure as this can capture the actual input to the structure via the pile foundation. Figure 4.5 shows the pile head response under the kinematic and inertial combined effect and also it compares the head responses with respect to the pile head response due to only the kinematic interaction effects. In this figure K and K+I stand for kinematic effect and combined kinematic and inertial effects respectively.

As seen in figure 4.5, the pattern of pile head motion under the combined kinematic and inertial combined effects is similar to the pile head response pattern under kinematic interaction effects for applied seismic excitations. However, amplification and a small phase lag can be observed here, due to the presence of structural mass attached to the pile head as it amplifies the lateral movement. Under the combined kinematic and inertial effects the pile is subjected to a maximum response of about 0.4m and as explained in section 4.5.1 kinematic interaction itself only gives a maximum response of about 0.3m. Hence the combined kinematic and inertial effect has increased the maximum pile head response by 33%, which infers that kinematic interaction effect is the dominant parameter in deciding the pile head response when subjected to seismic excitation.

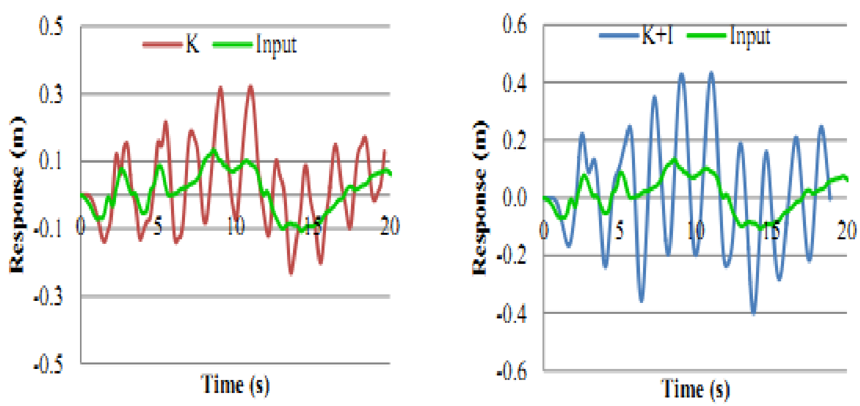


Fig 4.5 comparison of pile head response under kinematic interaction effects and kinematic and inertial combined effects in the case of El-cento earthquake.

4.5.3 Effect of pile size on pile head response

To examine the effect of pile size on pile head response, the size of the pile was increased from 0.25m x 0.25m to 0.5m x 0.5m, while keeping all the other parameters constant. The obtained results for the pile head response for the three seismic excitations were then compared with the corresponding pile head responses of the 0.25m x 0.25m pile for both kinematic interaction effects and combined kinematic and inertial effects.

4.5.3.1 Effect of Pile Size on Pile Head Response under Kinematic Interaction Effects

As seen from figure 4.6, effect of pile size in kinematic interaction under the conditions considered is insignificant. Responses of both piles in all these three cases are almost the same. Even though the general belief is that increase of pile should decrease the response, under the considered scenario, kinematic effects caused by the movement of the surrounding soil is the most significant factor in deciding the pile head response. This reduces the effect of increase in pile size with hardly any chance and hence does not result in a significant variation in pile head response.

4.5.3.2 Effect of Pile Size on Pile Head Response under Kinematic and Inertial Interaction Effects

Figure 4.7 shows the effect of pile size on pile head response when inertial effects are introduced in addition to kinematic effects. When the soil-pile system is under the El-Centro seismic excitation, increased pile size decreases the pile head response in an overall manner. However, the maximum pile head response is almost the same for both pile sizes. When subjected to Kobe earthquake, depending on the size of the pile, discrepancies can be observed during the first 15s, pile head response is almost the same during the rest of the period. However when the soil-pile system is excited with the Northridge earthquake, irrespective of the pile size, pile head response is identical during the entire period of excitation. Considering the results drawn from all three seismic excitations, generally it can be said that pile size does not considerably affect the pile head response under the kinematic and inertial combined effect.

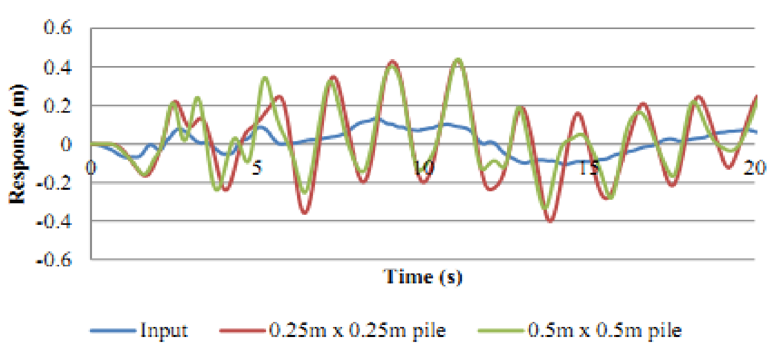


Figure 4.6: Effect of pile size on kinematic pile head response El-Centro earthquake

In general higher pile sizes are used to carry and transfer a higher super structure load. Therefore, when the pile size was increased from 0.25m x 0.25m to 0.5m x 0.5m, a study was carried out by increasing the superstructure mass from 100,000kg to 200,000kg to investigate the effect of higher superstructure mass on pile response. As seen in figure 4.8 in general pile head response pattern is almost the same for both masses but with an increase in magnitude and with a phase change. This is due to the increase of period of vibration caused by the increase of structural mass. However, the increase of maximum head response is increased by 36% for El-Centro, earthquakes.

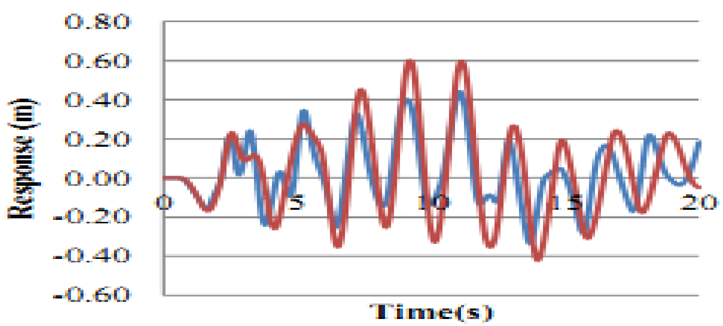


Figure 4.8: Pile head response variation due to the variation in structural mass El-Centro earthquake

4.6 Effect of soil stiffness on pile response

Kinematic soil-pile interaction problems generally deal with the deviation of pile motion with respect to the input motion. If the deviation of pile motion with respect to the input motion is negligible, it might be reasonable to carry out the analysis of piles according to pseudo-static analysis which neglects the kinematic soil-pile interaction effects. However, such scenarios are limited in real life applications and can be applied if the piles are short and embedded in relatively stiff soils. When long, slender piles are considered, kinematic interaction effects caused by the movement of surrounding soils can greatly affect the pile deflections along its depth. But the amount of deflection can also be influenced by the stiffness difference between pile and soil. This section presents the variation in seismic response of pile at different layers of the soil profile in which the pile is embedded in the time domain (Figures 4.9 to 4.13).

Input motion was selected as a baseline to compare the variation in pile response due to the stiffness difference between pile and soil. The pile response at the mid-depths of the soil layers is presented to illustrate the effect of soil stiffness on the pile response. In figures 4.9 to 4.13, “K” refers to the response due to kinematic interaction and “KI” refers to the response due to the combined kinematic and inertial effect. Also the results are shown for both pile sizes 0.25m x 0.25m and 0.5m x 0.5m. As seen in there, the portion of pile embedded in the softest layer shows the most significant deviation with respect to input motion under seismic excitations. However, as the stiffness of the soil increases with depth, the deviation of pile response with respect to input motion reduces and becomes almost zero in the stiffest layer. From these results it can be inferred that considering kinematic interaction effects is important for carrying out the seismic analysis of pile foundations when they are embedded in soft soils. However, this effect becomes negligible in the portions where the embedded soils are stiff. These results are valid for both pile sizes. Also in these results, significant changes between kinematic only and combined kinematic and inertial effect cannot be identified as the inertial interaction effects diminish with the depth.

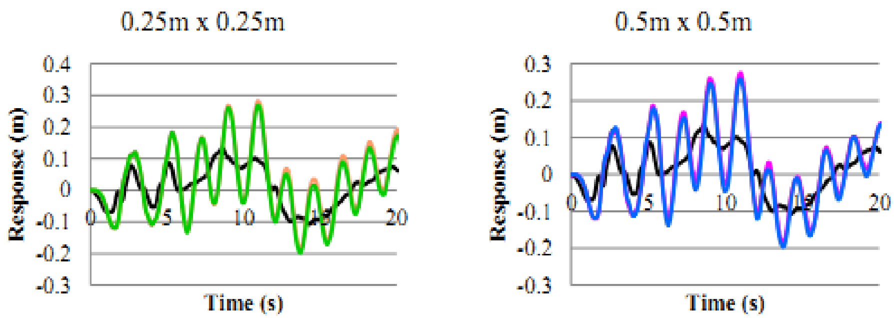


Figure 4.9 : Pile response at mid depth of layer -1 a) El-Centro earthquake

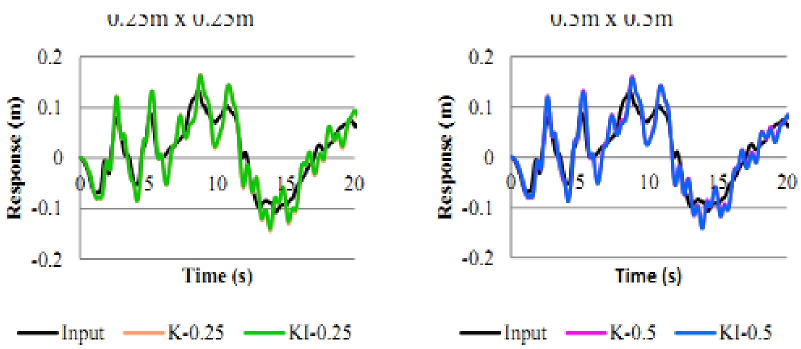


Figure 4.10: Pile response at mid depth of layer -2 El-Centro earthquake

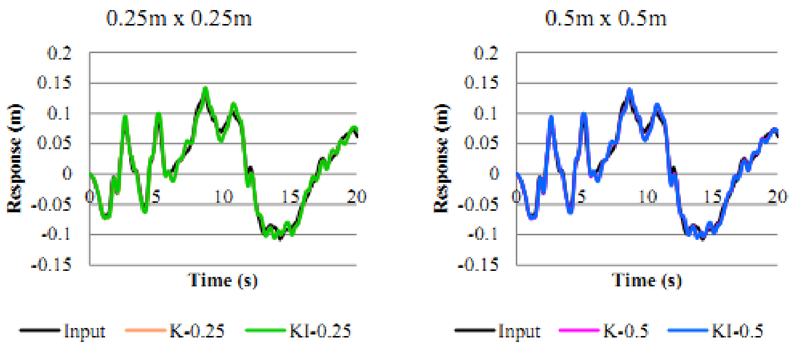


Figure 4.11: Pile response at mid depth of layer -3 El-Centro

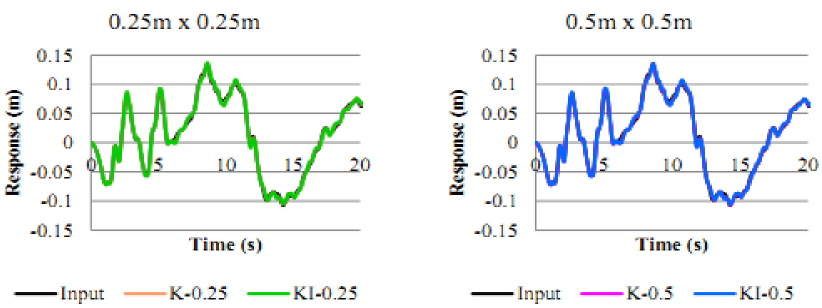


Figure 4.12: Pile response at mid depth of layer – 4   El-Centro earthquake

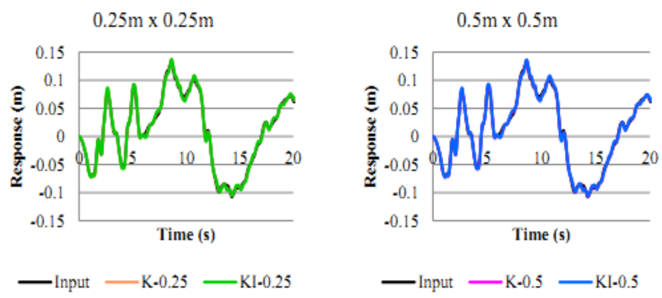


Figure 4.13: Pile response at mid depth of layer -5 El-Centro earthquakes

Furthermore, presence of soil layers with varying stiffness cause different movements in pile along its length in the horizontal direction when subjected to a seismic excitation. The following section describes the different deflection modes that a pile can undergo under seismic excitations, when they are embedded in layered soil profiles.

4.7 Pile Deflection Patterns

When long slender piles are embedded in layered soil, the deformation along the length of the pile can be governed by the stiffness of the surrounding soil layers as described in section 4.6 and results in differences in deformations along the length of the pile. These differences in deformation can cause different deflection modes in the pile which resemble forced vibration modes under the applied seismic excitation. Figures 4.14 to 4.16 show the different deflection modes obtained during the time domain analysis under the three different earthquakes at different times. As seen from figures 4.14 to 4.16, the stiff soil layers do not contribute in generating deflection modes as the relative deflections with respect to pile axis is negligible. Instead they provide the fixity to the foundation. When the pile was subjected to El-Centro earthquake 1st and 2nd modes are clearly visible. Even though 2nd and 3rd modes of deflections can be observed during the seismic excitation, 1st deflection mode is the dominant mode during the excitations with respect to kinematic interaction effects. However, incorporation of inertial interaction effects alters the deflection modes. This enables the more frequent occurrence of higher modes. Increase in pile size from 0.25m x 0.25m to 0.5m x 0.5m generally doesn't change the deflection mode shapes, but there are slight differences in deflection patterns. The 0.25m x 0.25m pile shows a more flexible behavior due to its higher aspect ratio when compared to 0.5m x 0.5m pile.

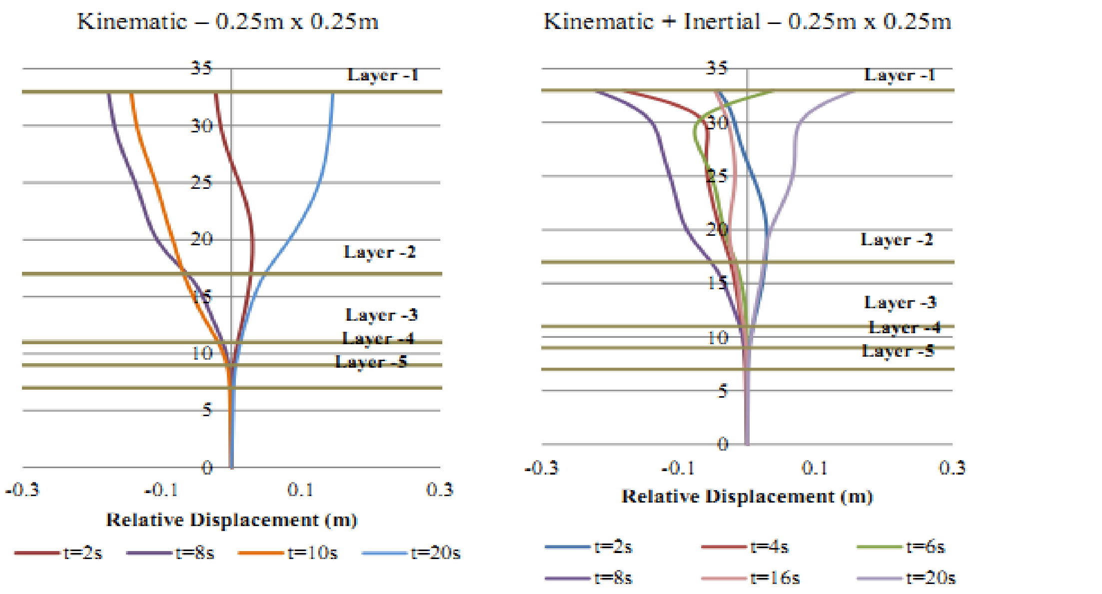


Figure 4.14: Deflection patterns of the pile when subjected to El-Centro earthquake

Figures 4.14 show the pile deflection patterns along the pile length in comparison with initial mass used (100,000kg) at different times during the time domain analysis. Changes to the deflection patterns are mainly observed in the upper part of the pile foundation where the inertial interaction effects become significant. The deflection patterns observed in two scenarios seems to be similar, while the higher super structure mass exaggerates the magnitude of the relative response. However, the time and the frequency of occurrence of different deflection mode shapes may be different due to the possible phase shift when in response due to the change of super- structural mass.

4.8 Maximum deflection along the pile length

Design of slender piles is usually governed by deflection and this should be maintained within the "permissible limit". Therefore ,displacement analysis is more appropriate than the stress analysis in designing of slender piles in most of the practical situations.in such cases, the maximum deflection a pile can undergo are of interest to designers to make sure the deflections

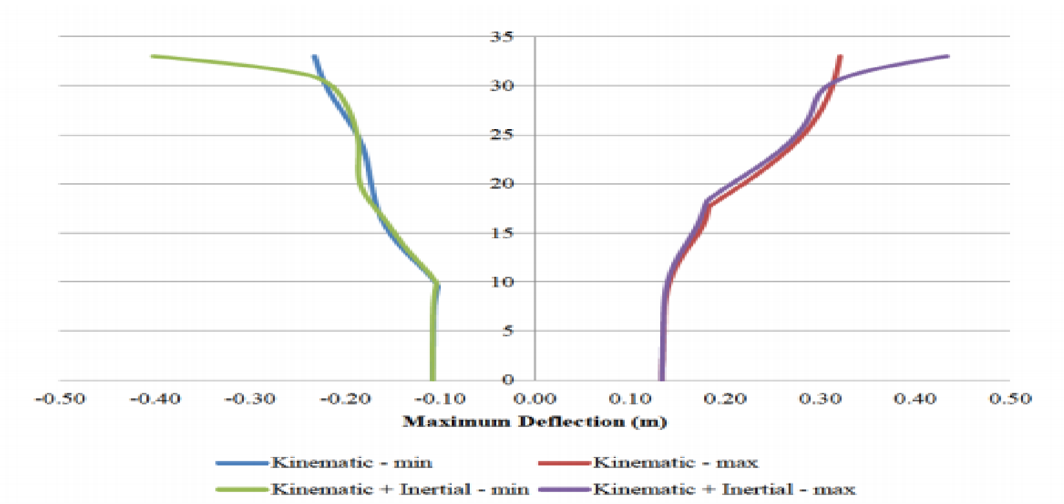


Figure 4.15: Deflection envelop along the pile length under El-Centro earthquake

According to the figure 4.15 maximum deflections along the pile length due only to kinematic effects is almost of similar to the maximum deflections due to the combined kinematic and inertial effect along the pile length excluding the top most 3m length pile. In the top 3m of the pile, maximum deflection is significantly affected by the inertial effects and hence increases the maximum deflection. However inertial effects decrease with depth of pile.

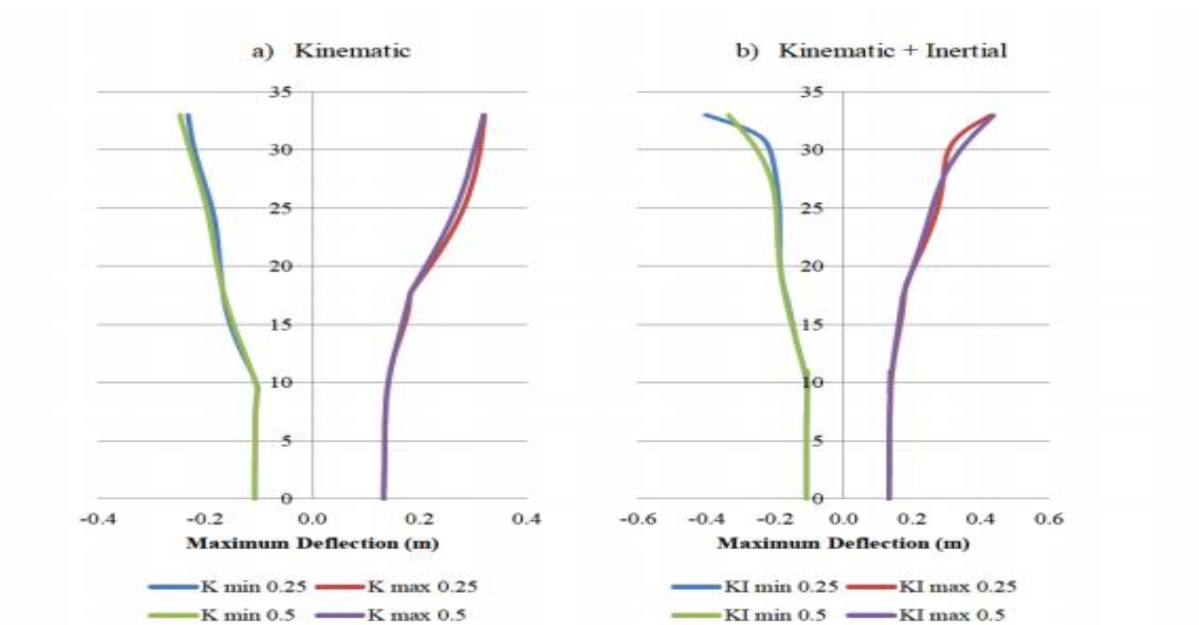


Figure 4.16: Effects of pile size on maximum deflection under El-Centro earthquake

Figures 4.16 show the effect of pile size maximum deflections. When the pile is subjected to only kinematic effects pile sizes have a negligible effect on the maximum deflections. When the inertial interactions are also incorporated in addition to kinematic effects, the upper portion of the piles show small deviations in the maximum deflection values with the smaller pile having higher maximum deflection at the pile head in general.

4.9 Summary

This chapter presented the study carried out to investigate the behavior of a pile foundation in a multilayered deep soil profile with a soft soil layer at the top. This included the investigation of both kinematic interaction effects and combined kinematic and inertial interaction effects under earthquake loadings. This study showed that pile head response can vary depending on the complex properties of the earthquake loadings. Furthermore the pile head response pattern is very much similar under both kinematic interactions effects and combined kinematic and inertial combined interaction effects. Inclusion of inertial effects amplified the head response due to the kinematic effects with a small phase lag. Under the considered soil profile properties, the response of the pile portion embedded in the soft soil layer varied mostly from the input motion and this variation reduced with the increase in soil stiffness as along the depth of the pile and the response of the portion of the pile embedded in the stiffest layers was almost same as the input motion. Due to this variation in pile response in different soil layers, different deflection modes are activated. These deflection modes also varied depending on whether kinematic interaction or combined kinematic and inertial combined interaction effects are considered.

Finally the maximum deflections along the pile depth were investigated for El-centro earthquakes under both kinematic and combined kinematic and inertial effects. Inclusion of inertial effects increased the maximum deflection in the upper part of the pile, its effect decreases with pile depth. Under the considered scenario, increase of pile size did not cause a significant impact on pile response.

Chapter 5

CONCLUSION AND RECOMMENDATION

5.1 Introduction

Earthquakes are one of the major forms of natural disasters that threaten lives and infrastructure. Loads imposed by earthquakes can be large and can cause catastrophic failure of structures. Understanding the seismic behavior of structures and structural components is therefore important in the design of structures that may be subjected to earthquakes. Among these structural components, foundations which are used to support the superstructures play a significant role as the failure of the foundation can ultimately lead to the failure of the whole structure. Among the different types of foundations used to support structures, pile foundations are given more attention as this type of foundations are much vulnerable to damage during earthquakes. During an earthquake pile foundations are subjected to lateral forces. Even though the primary function of a pile foundation is to carry and transfer the vertical loads from the superstructure, it has to withstand lateral forces due to seismic actions as well. The behavior of a pile foundation during an earthquake is not isolated as it is surrounded by the soil, which provides the lateral confinement to the pile foundation. Hence the seismic response of a pile foundation depends on the behavior of the surrounding soil as well which makes this soil- pile interaction problem a complicated one. Irrespective of the amount of research carried out in the area of soil-pile interaction problem, designing of a pile foundation still remains challenging, especially when it is embedded in soft soils. On the other hand the popular methods used to investigate the behavior of pile foundations under earthquakes such as Winkler method are believed to be inappropriate due to their inability to consider the soil's actual behavior as a continuum which is crucial in investigating pile behavior, especially under earthquake loads. On the contrary, FEM which is a continuum based method provides a more suitable approach for modeling and analyzing the soil-pile system. FEM however, requires a significant computational cost, which is one of the major concerns of using this method until recent times. Due to the advancement of technology, FEM is raising its popularity in solving soil-pile interaction problems and hence it is used in this project as the method of analysis.

5.2 Contribution from the project

In actual engineering practice, the design of pile foundations is based on the simplified method known as "Pseudo-Static Analysis". The major drawback in this method is the negligence of kinematic interaction effects on pile response, caused by the movement of surrounding soils, which is significant especially in vibration sensitive soft soils. This can lead to the under estimation of pile response under a seismic excitation which is often the cause for the pile failures under such circumstances. Post-earthquake field observations demonstrated the importance of incorporating kinematic effects in determining pile response when subjected to seismic loads. Design codes such as the Euro code mention that kinematic effects should be considered when designing pile foundations. However, neither a deterministic method nor other validated techniques are currently available to determine the kinematic interaction effects on response of pile foundations under a seismic excitation.

This research develops and applies a novel technique to determine the pile response under seismic loads, using the FE method which is known to be reliable in pile analysis. The developed method of analysis was then used to investigate the behavior of pile foundations in deep multilayered soil profiles with a vibration sensitive soft soil layer which are typically found in marine environments.

The main findings of this research are listed below.

1. This research has developed and applied a comprehensive dynamic computer simulation technique to study the response of piles subjected to seismic excitation. This technique has the capability to capture both the kinematic and inertial effects on the pile response. These effects are often neglected in the pile design process, but important in soil profiles with soft soils.
2. The modeling techniques can be used in actual engineering practices and also in further research on soil-pile interaction problems.
3. The time domain analysis carried out in the present study shows its capability in capturing important parameters such as maximum pile response, pile deflection patterns, possible permanent deformations, kinematic and inertial interaction effects, all of which provide useful

information in design processes.

4. According to the results, presence of stiff soil layers themselves does not significantly contribute to the kinematic interaction effects. If the stiff soil layers are present at the base of the profile, the pile response almost follows the input motion. However, if the stiff soil layer is present over a soft soil layer, the pile response does not follow the input motion. It is influenced by the underlying soft soil layer, but the pile response variation within the stiff soil layer remains negligible. As the stiffness of the soil decreases, pile response varies significantly from the input motion, even if only kinematic effects are considered.
5. Considering inertial effects in addition to kinematic effects amplifies the pile head response with a phase lag.
6. When analyzing a superstructure subjected to seismic loads, the normal engineering practice is to use the original input motion at the base of the structure. However, this research implies the significance of modifying the input motion instead of using the original seismic excitation as the actual input to the superstructure depends on the soil-pile interaction effects. In this case it is important to consider both kinematic and inertial effects as both have a significant influence on pile head response.
7. Results show that, the presence of a deep soft soil layer does not always increase the pile response and the presence of a thin soft soil layer does not always guarantee a reduction in pile response. This suggests that pile response is also influenced by the compliance of frequency content of input motion with the natural frequency content of the soil profile.
8. Presences of a soft soil layer at the top of the profile cause a pile foundation to undergo different forced vibration modes during a seismic excitation due to the lack of lateral support provided by the soft soil layer. However, if the soft soil layer is overlaid by a hard layer, complex deformation patterns are not visible in the pile within the soft soil layer as the pile is dragged by the upper hard layer.
9. Inertial interaction effects cause piles to have increased response in the upper part of the pile and hence maximum response due to combined kinematic and inertial effect is higher than the maximum response due to kinematic effect only in the upper part of the pile. As we go down the pile length, inertial effects diminish rapidly and the maximum response under both combined inertial and kinematic effects and kinematic effects only give the same

maximum response in the lower part of the pile. Furthermore, under some excitations an intermediate region can be identified where, the combined effect lowers the response when compared to the response due only to the kinematic interaction effects.

10. Increase of pile size does not have a significant effect on pile response under Kinematic interaction effects only. However, increase of superstructure mass with the increase of pile size exaggerates response due to combined effect. However, as only a limited amount or analyses were carried out in this case, further investigation is necessary to make a firm conclusion.
11. According the studies carried out with different soil profiles, it can be concluded that pile behavior is very unique. It is greatly influenced by the soil profile, thickness of the soft soil layers and frequency content of the input motion.

5.3 RECOMNDATION

- This study used mostly one standard pile size to investigate the behavior of deep pile foundations embedded in soil profiles with a soft soil layer. Further research can be carried out using different pile sizes and hence different inertial forces exerted by the superstructure.
- This research was limited to investigating the behavior of single free head piles.
- However, piles are generally built as groups. This will lead to the group effect and also a fixity condition to a certain extent. Therefore investigation of pile group behavior is suggested as further research.
- This study did not consider the time dependent behavior of the surrounding soils. Therefore, further research can be done introducing time dependent material properties to investigate the pile behavior under seismic excitation.

Reference

1. Yoshimoto, N., Hydo, M., Takahashi, N., Yamamoto, Y., Kimura, S., Earthquake Response for Stratified Deposits Composed of Clay and Sand LAyer by On Line Psueudo-Dynamic Response Test. in 13th World Conference on Earthquake Engineering. 2004. Vancouver, B.C., Canada

2. Wilson, D.W., Soil-Pile-Structure Interaction in Liquefying Sand and Soft Clay, in Department of Civil & Environment Engineering. 1998, University of California at Davis.

3. Raffaele, D.L., Alessandro, M. , George, M., Selection Criteria For Pile Diameter Ineismic Areas, In Second International Conference On Performance-Based Design In Earthquake Geotechnical Engineering. 2012: Taormina (Italy).

4. Tokimatsu, K., Suzuki, H., Sato, M., Effects of Inertial and Kinematic Interaction on Seismic Behavior of Pile with Embedded Foundation. Soil Dynamics and Earthquake Engineering, 2005. 25: p. 753-762.

5. Eurocode-8, Part 5 - Foundations, retaining structures and geotechnical aspects.1999.

6. Mylonakis, G., Gazetas, G., Seismic Soil-Structure Interaction: Beneficial or Detrimental? Journal of Earthquake Engineering, 2000. 4(3): p. 277 - 301.

7. Kavvads, M., Gazetas, G., Kinematic seismic response and bending of free-head piles in layered soil. Géotechnique,, 1993. 43(2): p. 207 –222

8. Balendra, S., Numerical Modeling of Dynamic Soil-Pile-Structure Interaction, in Department of Civil and Environmental Engineering. 2005, Washington State University.

9. Whitman, R.V., Analysis of Soil-Structure Interaction: State-of -the-Art Review. Vol.19. 1972: Massachusetts Institute of Technology, Soils publication

10. Mamoon, S.M., Ahmad, ., Seismic Response of Piles to Obliquely Incident SH, SV,and P Waves Journal of Geotechnical Engineering, 1990. 116(2): p. 186-204.

11. Gazetas, G., Seismic response of end-bearing single piles. International Journal of Soil Dynamics and Earthquake Engineering, 1984. 3(2): p. 82-93.

12. Boulanger, R.W., Curras. C. J., Kutter, B. L., Wilson, B. W., Abgharu, A., Seismic Soil-Pile-Structure Interaction Experiments and Analyses. Journal of Geotechnical and Geo environmental Engineering, 1998. 125(9): p. 750-759.

13. D.Basu, Salgado, R., Prezzi, M.,Analysis of Laterally Loaded Piles in Multilayered Soil Deposits. 2008, Indiana Department of Transportation and Purdue University: Indiana.

14. Terzaghi, N., Evaluation of coefficients of subgrade reaction. Geo technique, 1955.5(4): p. 297 326.

15. Bowles, J., Foundation analysis and design. 1997: McGraw-Hill.

16. Hetényi, M., Beams on elastic foundation. 1946: University of Michigan Press.

17. Novak, M.,Dynamic Stiffness and Damping of Piles. Canadian Geotechnical Journal,1974. 11: p. 574-598.

18. Nogami, T., Novak,M., Coefficients of Soil Reaction to Pile Vibration. Journal of the Geotechnical Engineering Division, 1980. 106(5): p. 565-570

19. Nogami, T., Konagai, K., Time Domain Flexural Response of Dynamically Loaded Single Piles. Journal of Engineering Mechanics, 1988. 114(9): p. 1512-1525.

20. Nogami, T., Otani, J., Konagai, K., Chen, H., Nonlinear Soil-Pile Interaction Model for Dynamic Lateral Motion. Journal of Geotechnical Engineering, 1992. 118(1): p. 89-106.

21. Naggar, M.H.E., Novak,M., Nonlinear Analysis for Dynamic Lateral Pile Response. Soil Dynamics and Earthquake Engineering, 1996. 15(4): p. 233-244.

22. Naggar, M.H.E., Bentley,K. J., Dynamic analysis for laterally loaded piles and dynamic p-y curves. Canadian Geotechnical Journal,, 2000. 37(6): p. 1166-1183.

23. Mostafa, Y.E., Naggar, M.H.E., Dynamic Analysis of Laterally Loaded Pile Groups in Sand and Clay Canadian Geotechnical Journal, 2002. 39(61358-1383).

24. Liyanapathirana, D.S., Poulos, H. G., Seismic Lateral Response of Piles in Liquefying Soil Journal of Geotechnical and Geoenvironmental Engineering, 2005. 131(12): p. 1466-1479.

25. http://www.camineros.com/docs/cam035.pdf.

26. Deng, N., Ma, Y., Bridge Engineering Handbook. 2000: CRC Press.

27. Poulos, H.G., Behavior of laterally loaded piles: I – single piles. Journal of Soil Mechanics and Foundation Division (ASCE), 1971. 97(SM5): p. 711-731.

28. Yegian, M. , Wright, S.G., Lateral Soil Resistance Displacement Relationships for Pile Fundation in Soft clays, in Offshore Technology Conference. 1973: Houston,Texas.

29. Desai, C.S. , Appel, G.C.,. 3-D Analysis of laterally loaded structures. in 2nd

International Conference on Numerical Methods in Geomechanics (ASCE) . 1976.Blacksburg.

30. Kuhlemeyer, R.L., Static and Dynamic Laterally Loaded Floating Piles. Journal of the Geotechnical Engineering Division, 1979. 105(2): p. 289-304.

31. Abques 6.13.1 user manual

32. Wu, G., Finn, W.D.L., Dynamic Elastic Analysis of Pile Foundations Using Finite Element Method in the Frequency Domain. Canadian Geotechnical Journal, 1997. 34:p. 34-43.

33. Wu, G., Finn, W.D.L., Dynamic Nonlinear Analysis of Pile Foundations using Finite Element Method in the Time Domain. Canadian Geotechnical Journal, 1997. 34: p.44-52.

34. Cai, Y.X., Gould, P.L., Desai, C.S., Nonlinear analysis of 3D seismic interaction of Soil-pile-structure systems and application. Engineering Structures, 2000. 22(2): p.191-199.

35. Bentley, K.J., Naggar, M.H.E., Numerical Analysis of Kinematic Response of Single Piles. Canadian Geotechnical Journal, 2000. 37(6): p. 1368–1382.

36. Maheshwari, B.K., Truman, K. Z., Naggar, M. H., Gould, P. L., Three-Dimensional Finite Element Nonlinear Dynamic Analysis of Pile Groups for Lateral Transient and Seismic Excitations. Canadian Geotechnical Journal, 2004. 41(1): p. 118-133.

37. Maheshwari, B.K., Truman, K. Z., Naggar, M. H., Gould, P. L., Three-Dimensional Nonlinear Analysis for Seismic Soil-Pile-Structure Interaction. Soil Dynamics and Earthquake Engineering, 2004. 24(4): p. 343-356.

38. Lysmer, J., Kuhlemeyer,R.L., Finite dynamic model for infinite media. Journal of the Engineering Mechanics Division, ASCE,, 1969. 95(EM4): p. 859-877.